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Sewage Research

EFFECT OF TEMPERATURE AND SEEDING ON HYDROGEN SULPHIDE FORMATION IN SEWAGE *

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Hydrogen sulphide production in sewers is an important factor in sewer disintegration, and odors emanating from sewer lines, catch basins, pumping stations, and sewage treatment plants. It has previously been shown [5] that incubation of domestic sewage under anaerobic conditions shows the presence of a very small proportion of combined sulphides as compared with volatile sulphide (hydrogen sulphide) throughout the course of decomposition. The experimental results pointed to the presence of intermediate organic sulphur compounds as sources of hydrogen sulphide. Investigations showed also that a number of sulphates, including sodium, ammonium, magnesium, calcium and ferrous sulphates were potential sources of hydrogen sulphide. With additions of ferric sulphate there was a distinct retardation in the decomposition of the sulphate.

Three questions were raised in connection with these experiments, (1) what is the effect of temperature, (2) how much faster will the sulphates be decomposed when a concentration of sulphur splitting organisms is present and (3) what is the effect of some substances commonly present in sewage.

The amount of work done on the factors affecting odor production is meager. In connection with studies on changes of sulphur compounds during sewage treatment, Buswell and Elder [2] incubated sewage in stoppered bottles at 9, 22 and 37° C. and showed that the rapidity of sulphide production increased with the increase in temperature. Porter and Cresswick [4] working with slaughter-house sewage found that at laboratory temperatures it took 24 hours to obtain the same results as in 6 hours at 32° C. and in 1 hour at 37° C.

Goudey [3] working on the control of hydrogen sulphide production in sewers by chlorination, found that sewage flowing through an 18-inch force main, having an average detention period of 12 hours, developed 22.5 p.p.m. hydrogen sulphide. The sulphate splitting organ-

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isms apparently started their action along the sides of the sewer when the sewage was 2 to 3 hours old. Hydrogen sulphide at first was formed at a high rate, faster than it could be released, for as the sewage continued to flow, H_2S production apparently became constant, while the sulphides were reoxidized to almost the initial content of sulphates in the lower regions of the sewer.

Methods

(*a*) *Sampling.*—Sewage used in the experiments was taken from the influent channel of the Plainfield, N. J., sewage disposal plant. It was taken at approximately 11 A.M., since sewage of average daily composition enters at that time. This sewage is entirely domestic, averaging about 220 p.p.m. suspended solids. It is approximately six hours old when it reaches the plant. However, the sewer leading to the plant is designed to give a rapid flow with no deposition of solids. All samples were incubated in 500 c.c. rubber-stoppered glass bottles.

(*b*) *Hydrogen Sulphide Determination.*—Two methods were used to determine hydrogen sulphide. One, that of Almy [1] who used it to determine hydrogen sulphide in proteinaceous food products, and was employed when the H_2S content did not exceed one or two p.p.m. The other method was the common one in which the hydrogen sulphide is allowed to react with standard iodine solution and the excess titrated with standard thiosulphate solution.

In both cases the hydrogen sulphide was driven out of the sewage by carbon dioxide to remove the danger of interfering substances. This was accomplished by building an aerator from large-bore glass tubing, the size depending on the magnitude of the sample to be aerated. An alundum filter disc was fastened in the lower end of the tube. Carbon dioxide, led in under this plate, diffused through the plate, giving an even flow of fine bubbles throughout the liquid. A glass tube at the top of the aerator led the gases to a ten-bulb absorption tube where hydrogen sulphide was either absorbed in zinc acetate solution or allowed to react with iodine solution.

(*c*) *Determination of Sulphates.*—Sulphates were determined by the gravimetric method by precipitation as barium sulphate and weighing after ignition.

(*d*) *Determination of pH.*—pH was determined colorimetrically.

Results

Temperature.—Glass bottles of 500 c.c. capacity were filled with sewage and closed with rubber stoppers. These bottles were divided into sets and placed in three incubators, each kept, thermostatically, at a

definite temperature. Another set was placed in an electric refrigerator in which the temperature was maintained at 7.0° C. The incubation temperatures were 21° C., 30° C. and 37.5° C. One of the bottles was removed daily, or at such time as determinations were desired and the hydrogen sulphide or volatile sulphide together with the combined sulphide and pH were determined. The volatile sulphide content was determined by aeration without the addition of acid while the combined sulphide was determined on the same sample after addition of acid. Sulphates were determined in the sewage at the beginning and end of the experiment.

The curves of total sulphide production at the various temperatures are shown in Figure 1. These curves show in all cases that there was

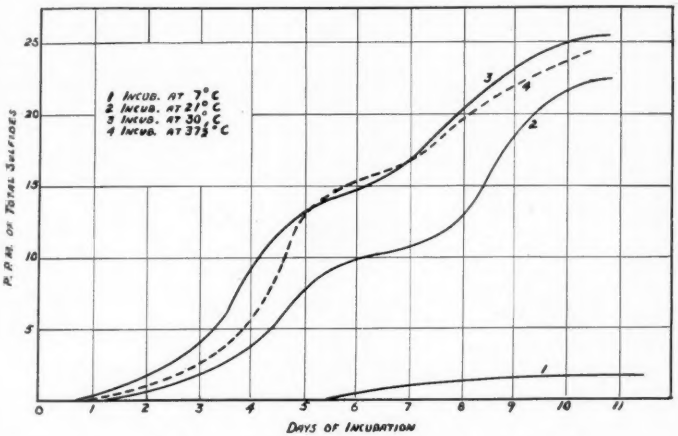

FIG. 1.—Effect of temperature on the production of sulphides in sewage.

little production of sulphide in the first two days. Intensive sulphide production began between the third and fourth days in the case of the three higher temperatures, namely, 30° C., 37.5° C. and 21.0° C. Another point of intensive sulphide production came about the eighth day, after which the curves flattened, showing a falling off of production. Sulphide production was practically complete in ten days in all cases, except at 7° C.

Sulphide production at 7° C. was comparatively small, amounting to less than 2 p.p.m. in eleven days. There was quite a difference in rate of production of sulphide from samples incubated at 21° C. and 30° C. The samples incubated at 37.5° C. did not show any great increase over those at 30° C. In fact the sulphide production was somewhat retarded

in the first case at 37.5° C. incubation over the 30° C. incubation temperature.

Combined sulphides remained at a low value and show no definite variations. Values for these are given in Table I.

TABLE I.—*Effect of Temperature on Sulphide Production*
Results in Parts per Million

Days	Incubation Temp. 37.5° C.			Incubation Temp. 30.0° C.			Incubation Temp. 21.0° C.			Incubation Temp. 7.0° C.		
	P.p.m. Vol. Sulphide	P.p.m. Comb. Sulphide	P.p.m. Total Sulphide	P.p.m. Vol. Sulphide	P.p.m. Comb. Sulphide	P.p.m. Total Sulphide	P.p.m. Vol. Sulphide	P.p.m. Comb. Sulphide	P.p.m. Total Sulphide	P.p.m. Vol. Sulphide	P.p.m. Comb. Sulphide	P.p.m. Total Sulphide
1	0.1	0.24	0.34	0.03	0.07	0.1	0.012	0.012	0.024	0.005	0.017	0.022
2	0.53	0.47	1.00	0.47	1.06	1.53	0.35	0.24	0.59			
4	2.65	0.59	3.24	4.3	3.9	8.2	2.64	0.59	3.23	0.013	0.03	0.043
5	13.2	1.9	15.1	13.2	1.08	14.3	7.3	1.6	8.9			
6	12.8	0.7	13.5	12.8	0.7	13.5						
7	13.3	0.4	13.7				8.1	1.5	9.6	0.46	0.54	1.00
11	24.0	1.2	25.2	24.0	1.9	25.9	21.6	0.9	22.5	0.7	1.00	1.7

Sulphates remained in the samples after eleven days of incubation in all cases as shown in Table II. The amount of sulphide produced

TABLE II

Temp. of Incubation	37.5° C. P.p.m. SO_4	30.0° C. P.p.m. SO_4	21.0° C. P.p.m. SO_4	7.0° C. P.p.m. SO_4
Beginning	76.6	76.6	76.6	76.6
End	17.1	12.9	13.5	65.6
Decomposed	59.5	63.7	63.1	11.0

was greater than could be accounted for from the sulphates decomposed. Although this excess only amounted to 3 or 4 p.p.m. in most instances it points to some other form of sulphur as its source.

pH determinations did not show any great variation. Values for these are given in Table III.

Seeding.—The effect of the addition of stale solids on the production of hydrogen sulphide was studied by adding to fresh sewage, stale solids

TABLE III

Days	37.5° C. pH	30.0° C. pH	21.0° C. pH	7.0° C. pH
0	7.5	7.5	7.5	7.5
1	7.1	7.1		7.3
3	7.1	7.0	7.1	7.3
4	7.0	7.0	6.9	
6	7.2	7.0	7.1	7.1
11	7.2	6.9	7.1	7.1

of approximately 3.5 per cent concentration on a dry solids basis and with a pH of 5.5. Two separate experiments were carried out. All incubations were made at 26° C. For the first, or preliminary experiment, 0.4 per cent of this stale sludge, several months old, was added to the sewage. In the second experiment different quantities of this sludge were added, namely 0.05, 0.1 and 1 per cent.

Figures 2, 3 and 4 show the results of these experiments. In Figure 2 we have the plot of the sulphide production in the seeded liquid and

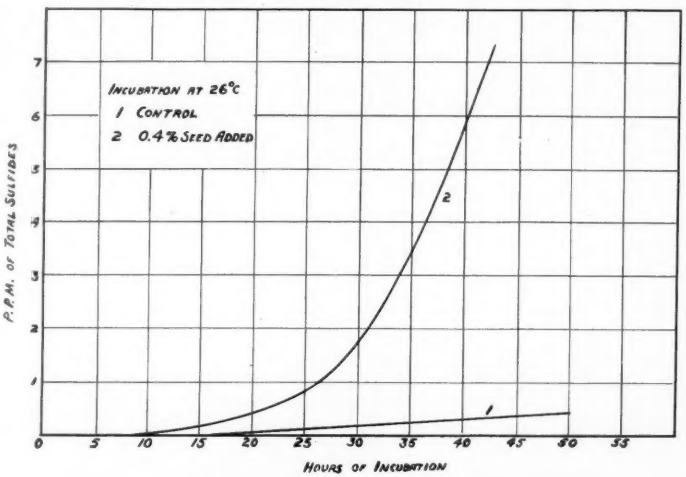

FIG. 2.—Effect of seeding on sulphide production.

the control for the first few hours. Sulphide production in the seeded liquid increased rapidly in the first few hours, the sulphide content greatly exceeding that of the control. In 43 hours the total sulphide

content of the seeded sewage was 7.4 p.p.m. while that of the control was only 0.4 p.p.m.

Figure 3 shows the daily values for total sulphide content in the case of the control and the seeded mixture. Sulphide production in the seeded mixture exceeded that of the control in all cases. A steady increase in rate of production began on the second day in the seeded mix-

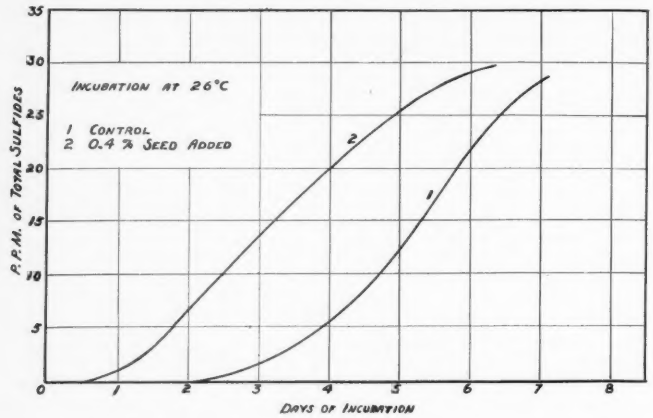

FIG. 3.—Effect of larger quantities of seed on sulphide production.

ture, but not until nearly the fifth day of incubation in the unseeded sewage.

Figure 4 shows the daily values of sulphide content in the sewage where different quantities of the seeding substance have been added. Sulphide production started much earlier in seeded than in unseeded mixtures. However, there was a rapid rise in the control values on the third day which caused the curve for the 0.05 per cent seeded mixture to fall below it at that time. The values for the tests in which seeding consisted of 0.1 and 1 per cent additions were always greater than those of the control. In the curve where 1 per cent seed was used there was an extremely sharp rise in the concentration of sulphides from the beginning of incubation, amounting to 5 p.p.m. in twenty-four hours.

pH values as shown in Table II were lower for the seeded mixtures due to the greater rapidity of decomposition and also to the addition of the sludge, which had a pH of 5.5. This variation in pH did not amount to more than 0.2 increment of pH at most and did not fall below a pH of 6.8. This would have no great effect on bacterial action.

The sulphide production from two representative samples taken on January 25 and April 22, is plotted in Figure 5. The temperatures of

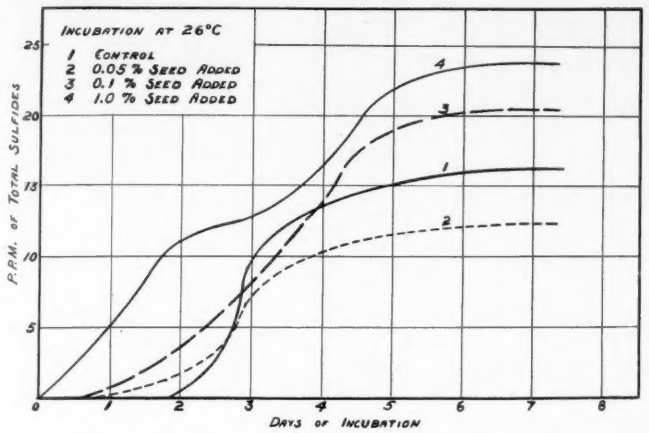

FIG. 4.—Effect of different amounts of seeding on sulphide production.

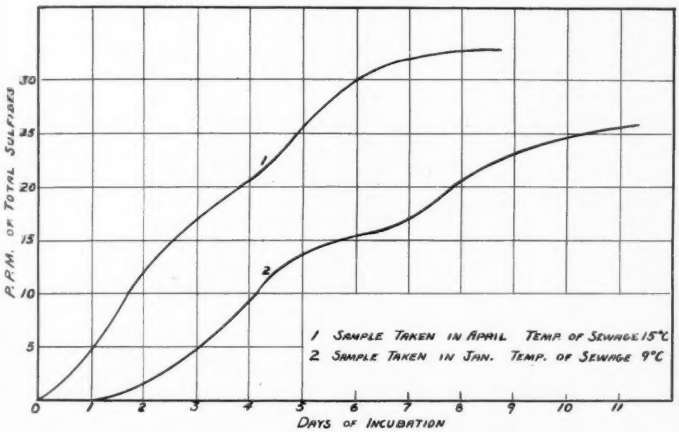

FIG. 5.—Seasonal effect on sulphide production.

the sewage influent were 9° C. and 15° C., respectively. A sample taken during the spring in comparatively warm weather decomposed very rapidly. The sulphide content in the sample collected in April rose to more than 4 p.p.m. in 24 hours while that of the sample taken in January was still negligible in this time of incubation.

Chemicals.—The effect of the nature of compounds containing sulphur has been reported elsewhere,[5] but some of our unpublished results,

obtained on the effect of dextrose and sodium bicarbonate, are of interest, since these substances are often present in sewage and may be a factor in stimulation of odor production. The results show that neither dextrose nor sodium bicarbonate caused any increase in the rate of sulphide production. These compounds were added up to a concentration of 100 p.p.m. Figure 6 shows the results in the control and the sewage to which dextrose has been added. Both curves coincide up to

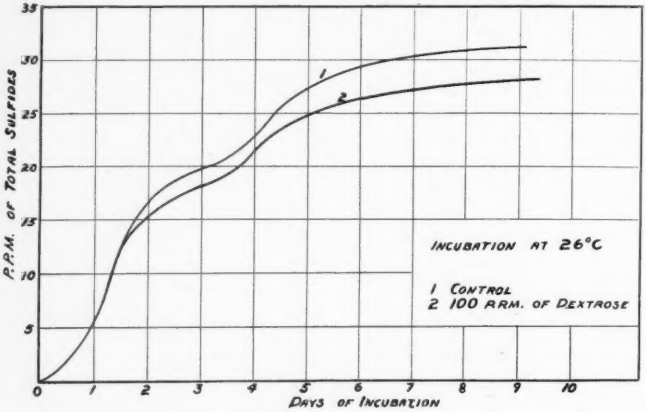

Fig. 6.—Effect of dextrose on sulphide production.

the second day after which the one for the dextrose containing mixture falls slightly below that of the control. The pH for the dextrose containing mixture was 0.2 to 0.3 increments of pH below that of the control.

Figure 7 shows that the sulphide production in the sewage to which bicarbonate was added was approximately the same as in the control.

Figure 8 shows the effect of 20 p.p.m. $FeCl_3$ and 10 p.p.m. $FeCl_3$ additions on the volatile sulphide content of sewage during incubation. There was no great effect in the lowering of this volatile sulphide content until after the fourth day. In fact, before this time there was an increase in the volatile sulphide content in the cases where ferric chloride had been added. Decomposition during the first four days was more rapid in the samples to which ferric chloride was added.

It might be added that a yellow coating was found on bottles, particularly in the neck, which contained sewage incubating for some time. The coating had the appearance of elementary sulphur but no attempt was made to identify it.

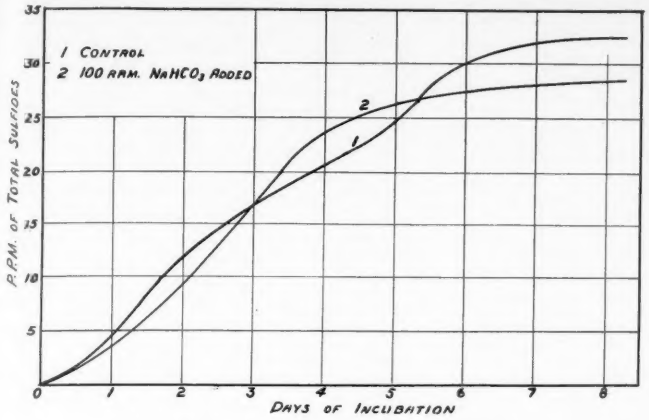

FIG. 7.—Effect of $NaHCO_3$ on sulphide production.

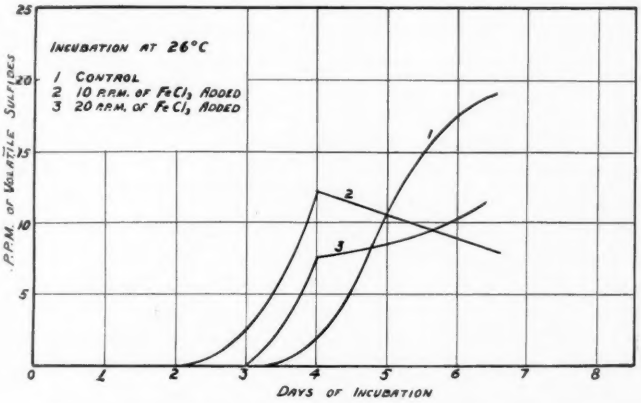

FIG. 8.—Effect of $FeCl_3$ on volatile sulphide production.

DISCUSSION

Experimental results showed that fresh sewage does not form hydrogen sulphide readily for several days. In fact, sewage which has not been allowed to come in contact with foreign material or putrescible material requires three to four days before it produces hydrogen sulphide to any great extent. Sewage used in these experiments which was collected during the winter months might come under the classification of fresh sewage, since little putrefaction occurs in the sewers at this tem-

perature. These results agree admirably with those of Porter and Crosswick [4] who worked with synthetic mixtures of slaughter-house waste.

Temperature has an important effect on the rate of hydrogen sulphide production. This is to be expected, since sulphide production is a direct result of bacterial action. Because of this fact sulphide production ceases rather sharply below a definite temperature, due to the failure of the organism to develop. Above this temperature, which is in the neighborhood of 7° C., there are not large differences in the rate of hydrogen sulphide production with increase or decrease in temperature. Experimental evidence points to the fact that there is abundant production of sulphides at 21° C., 30° C. and 37.5° C. Also, the rapidity of production is somewhat higher at 30° C. than at 21° C., but between 30° C. and 37.5° C. there is a slight retardation in rate.

It is well known that hydrogen sulphide is liberated in sewers and treatment works when the sewage is but a few hours old. Seeding seems to be the only explanation of this phenomenon which occurs in practice, but not in the laboratory, unless seed is introduced artificially. Whether it be due to the introduction of sulphate reducing organisms or to an available food source for these bacteria is mere conjecture. It is probably a combination of these two factors which brings about sulphide production in such a short time.

The seed used for experiments presented here could not be considered ideal. This substance, although highly putrescible, was well past the stage of active sulphate decomposition. Therefore, while showing a tremendous effect in bringing about sulphide production, it evidently did not contain as great numbers of sulphate reducing organisms as would be present in any substance in which rapid reduction of sulphates was taking place. An examination of the curves in Figure 4 shows that 1 per cent seed was required to bring about sulphate reduction at a rapid rate in the first few hours. This evidence indicates that the addition of highly putrescible substances requiring large quantities of oxygen from the sulphates for stabilization was paramount.

Putrescible material collects in many places in trunk sewers, sewers leading to a plant and in the plant itself. Such material continually diffusing into the sewage could be classified as an ideal seeding material. The sewage flowing over it constantly replenishes by deposition quantities of fresh material and by diffusion an abundant supply of sulphates. Therefore we have a deposit in which sulphate decomposition is taking place rapidly and consequently it contains large numbers of sulphate reducing organisms. Proof of this is shown in Figure 5, which is a comparison of the rate of decomposition of samples taken at the end of a sewer at different times of the year. In winter the flora in the sewer due to the low temperature has not had a chance to develop and

sulphide production is very slow. The sample taken in the spring gives off hydrogen sulphide rapidly from the beginning, showing that seeding has taken place in the sewer.

When the two factors seeding and temperature are combined, as will inevitably happen in sections of the country where comparatively large quantities of sulphates are present in the original water and the temperature of the sewage is high during a large part of the year, the results will be a rapid and continuous production of hydrogen sulphide. The production will be increased and the rate of production accelerated with subsequent odor and sewer disintegration troubles. It is also apparent that organic trade wastes containing sulphur, discharged at higher temperatures into sewers where a coating is present containing sulphate splitting organisms, will produce H_2S rapidly.

Any attempt to prevent the collection of putrescible material in sewage treatment works is decidedly a step toward odor control. Sewers can be designed to give flow velocities which minimize deposits. The design of catch-basins and pump-wells to prevent deposition of solids or their elimination as far as possible, would destroy a fruitful seeding source. Frequent removal of solids from settling tanks and the efficient operation of settling tanks would also help. Since coarse solids are readily deposited, preliminary screening or settling of sewage, where practicable, before it enters long trunk sewers may serve to reduce those deposits in the sewer and consequently hydrogen sulphide production.

Dextrose as shown in Figure 6 is evidently not a source of food for sulphate-reducing organisms since it causes no stimulation in sulphide production.

Sodium bicarbonate in the concentration used did not stimulate or retard sulphide production.

The ferric chloride added to the sewage initially, at a concentration of 10 to 20 p.p.m. as shown in Figure 8, increased the rate of hydrogen sulphide production during the first four days. This initial stimulation is of decided interest. It is well known that small quantities of iron compounds stimulate bacterial action under certain conditions. When more iron salt is added the organic material is flocked out and the sulphur compound temporarily locked up. In the second place, sulphur compounds are precipitated by the addition of iron salts, but this effect is slow with small quantities and rapid with larger amounts. In the experiments under discussion comparatively small quantities of $FeCl_3$ were added. In the mixture receiving 20 p.p.m. the production of H_2S was retarded and less in quantity than in the mixture receiving 10 p.p.m. $FeCl_3$. The effect of $FeCl_3$ would naturally vary with the strength and character of the sewage.

In the experiments reported, precipitation began after 4 days incubation and a reduction in volatile sulphide content is shown. This precipitation might take place in two ways. The first involves the presence of hydrogen sulphide in the sewage when ferric chloride is added and is according to the equation:

$$(1)\quad \begin{cases} (a)\ 2FeCl_3 + H_2S \rightarrow \mathbf{S} + 2FeCl_2 + 2HCl \\ (b)\ 2FeCl_2 + 2H_2S \rightarrow \mathbf{2FeS} + 4HCl \end{cases}$$

According to the second hypothesis the ferric iron is reduced by anaerobic bacterial action to the ferrous state and the precipitating reaction is as follows:

$$(2)\quad FeCl_2 + H_2S \rightarrow \mathbf{FeS} + 2HCl$$

The second hypothesis can be readily adapted to the conditions present in this experiment. Since the concentration of ferric chloride was low the iron was probably reduced to the ferrous state during the period when little precipitation took place.

When hydrogen sulphide is precipitated according to the first reaction we have three molecules of hydrogen sulphide precipitated for two molecules of ferric chloride added. The second equation gives us only one molecule of hydrogen sulphide precipitated for one molecule of ferric chloride added. From the standpoint of odor prevention it appears more economical therefore to add the ferric chloride to sewage containing hydrogen sulphide than to that in which H_2S has not yet been produced. This conclusion might lead to the practice of using $FeCl_3$ for odor control in sewers as soon as H_2S is formed, rather than an attempt to keep the sewage "fresh" or retard the growth of sulphur splitting organisms as practiced by the addition of chlorine. When chlorine is added the growth of the organisms is retarded, with subsequent prevention of odor production, whereas the addition of $FeCl_3$ would stabilize the organic matter temporarily and prevent rapid action by the organisms present. The partial precipitation of sewage in the sewer may have the additional benefit of improving settling of sewage at the treatment plant. However, if too much $FeCl_3$ is added and the velocity of the sewage is retarded anywhere along the line sedimentation may counteract the beneficial effects. Offhand, it would seem that prevention of bacterial sulphate reduction by chlorination is preferable to curing of odor by $FeCl_3$, but the answer will probably vary for different conditions. It is possible that both methods may be used side by side, but operation and relative cost will be the determining factor.

Communities which harbor industries discharging iron pickling liquors benefit, especially when such discharges can be made continuous, because the iron in pickling liquors is usually in the ferrous state.

The state of the iron accounts for the reduction of hydrogen sulphide production in sewage containing iron waste.

Summary

Studies of the production of sulphides by the reduction of sulphates were carried out by placing fresh sewage in bottles and incubating at different temperatures. Stale solids were added and the seeding effect determined. Other experiments consisted in the addition of certain chemicals as sodium bicarbonate, dextrose and ferric chloride. These experiments led to the following conclusions.

The two most important factors causing the production of hydrogen sulphide during sewage treatment are temperature and seeding. Seeding might be called the most important of these two because of the limited time required in the treatment of sewage. Were it not for seeding we would not have any nuisance due to hydrogen sulphide during this limited time.

To those factors we might add the concentration of solids since the oxygen demand of the organic material influences the amount of sulphates reduced and hence the amount of hydrogen sulphide produced. Therefore the concentration of sulphates above the minimum required for stabilization of organic material does not have any increasing effect on the amount of hydrogen sulphide present in sewage.

Inorganic sulphur containing compounds having higher ratios of sulphur to oxygen in their composition produce greater quantities of hydrogen sulphide.

Neither dextrose nor sodium bicarbonate stimulates the production of hydrogen sulphide.

Ferric chloride, in small quantities, is effective in reducing by precipitation the amount of volatile sulphide in sewage.

Since the bactericidal action of ferric chloride is negligible as compared with chlorine and relatively large quantities of this salt are required for stabilization of the organic matter, its value as a preventive for sulphide production when added in concentration of 10 to 20 p.p.m. can be questioned.

Conclusions

1. Fresh sewage does not produce sulphides in appreciable amounts (less than 1 p.p.m.) in from two to three days.

2. Sulphide production proceeds at a greater rate in sewage maintained at a temperature of 30° C. than when maintained at a temperature of 21° C. Production of hydrogen sulphide at 37.5° C. is at the same rate as at 30° C. Only small amounts of sulphides are formed at 7° C.

3. The introduction of stale solids to fresh sewage brings about decomposition at a rapid rate in a few hours, as indicated by the presence of sulphides.

4. Sewage collected at the end of a sewer line in spring decomposes more rapidly than that collected in winter when incubated at room temperature (20° C.), due to the development of flora in the sewer.

5. Sulphates present in sewage in greater amounts than required to satisfy the oxygen demand for stabilizing the organic material do not increase the rate of production of sulphides.

6. Dextrose at a concentration of 100 p.p.m. does not stimulate the production of hydrogen sulphide from sewage.

7. Sodium bicarbonate at a concentration of 100 p.p.m. does not increase the rate of sulphide production in sewage.

8. Ferric chloride at a concentration of 10 and 20 p.p.m. reduces the concentration of hydrogen sulphide in sewage after some time.

9. Stimulation of sulphide production was noticed initially when ferric chloride was added at a concentration of 10 to 20 p.p.m.

TABLE IV.—*Incubation at 26° C.*

Days of Incubation	Control pH	0.05% pH	0.1% pH	1% pH
0	7.4	7.4	7.3	7.3
1	7.3	7.2	7.2	7.1
2	7.4	7.2	7.1	7.0
3	7.0	7.0	7.1	7.0
5	7.0	7.0	6.8	6.9
7	7.0	6.9	7.0	6.9

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APPARATUS FOR THE DETERMINATION OF DISSOLVED OXYGEN IN SLUDGE-SEWAGE MIXTURES

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Research activities at the Stream Pollution Investigations Station of the United States Public Health Service in Cincinnati, Ohio, have recently been centered on the elucidation of one of the weakest links in the activated sludge process, namely, the troublesome condition, occasional or otherwise, of poor settlement generally designated as the "bulking" of the sludge.* Using a small experimental unit, the approach to this problem has been from the chemical, biological and physical or engineering viewpoints. In either case, it has appeared highly advisable to obtain accurate information regarding the dissolved oxygen content of the sludge-sewage mixtures. The development of the apparatus to be described in this paper was accordingly undertaken after the procedures then available had been shown to fail utterly in meeting the severe condition imposed by the presence of sludge.

The governing consideration in the determination of dissolved oxygen in such a highly putrescible material as activated sludge is the uncommonly high oxygen demand of the material itself. Using the apparatus described by Theriault and McNamee,[1] it can readily be shown that the oxygen demand of sludge-sewage mixtures drawn from aeration tanks may exceed 1 milligram per liter per minute and this figure may be multiplied by 5 or 10 when sludge drawn from clarification tanks is examined. At ordinary temperatures, the dissolved oxygen content of activated sludge will not exceed 9 milligrams per liter, and, generally speaking, it will be much lower. It is clear that time-consuming manipulations should be avoided in the examination of sludge-sewage mixtures for dissolved oxygen.

An artifice which is commonly used consists in eliminating most of the sludge by allowing it to settle. Tests for dissolved oxygen may then be made on the relatively clear supernatant liquor. Some of the sources of error in this procedure, particularly in the collection and handling of the sample, may be avoided by the use of the apparatus described by Küchler.[2] The procedure of allowing the sludge to settle is nevertheless impractical with "bulking" or poorly settling sludge. Even with "good" sludge the dissolved oxygen content of the supernа-

* These investigations are being conducted under the direction of Sanitary Engineer J. K. Hoskins. It is expected that further reports on various aspects of the work will be issued from time to time.

tant liquor should be appreciably reduced during the initial period of turbulence which precedes settling or by convection currents after settling begins. The interpretation of results is further complicated by surface aeration during settling and by the absorption of atmospheric oxygen in the transfer of the supernatant liquor, unless special apparatus is used.

The use of mercuric chloride is recommended by Konstantinowa [3] as an inhibitant of biochemical processes during tests for dissolved oxygen in the presence of activated sludge. Mercuric chloride is of doubtful efficiency as a sterilizing agent in activated sludge. Moreover, as shown by direct tests, the "immediate" or purely chemical oxygen demand is not appreciably affected by this reagent. It can also be shown that such a strong oxidizing agent as potassium permanganate will not prevent the loss of dissolved oxygen from freshly prepared dilutions of stale sewage (cf. Theriault [4]).

As modified by Theriault and McNamee,[5] the Winkler technique has been successfully applied to the determination of dissolved oxygen in the presence of relatively stable forms of organic matter, such as glucose, even in amounts up to 5000 p.p.m. (0.5 per cent). Reasonably accurate results were also obtained with freshly aerated peptone solutions, up to 600 p.p.m., and with partly oxidized sludge from artificial channels. Huge errors, however, were observed in experiments with unstabilized peptone solutions, and later the method was found to fail altogether in the presence of activated sludge.

From the foregoing survey of the subject it has not appeared that any purely chemical procedure could be used in the important marginal case where the dissolved oxygen content of a sludge-sewage mixture is 1 p.p.m. or thereabouts. The desideratum is evidently a method whereby dissolved oxygen can be separated from activated sludge in a few seconds, instead of in a few minutes. Such a method should provide a record of the momentary situation in the unstable equilibrium maintained by the constant air supply against the unsatisfied oxygen demand of the sludge. Physical methods for the extraction of gases from liquids were accordingly examined.

Extraction of Gases from Liquids

There is a wide choice in the selection of methods for the extraction of gases from liquids. For the purpose at hand, with time as the controlling factor, a method based on the injection of the sample into a highly evacuated space has appeared to be the most practical. The scrubbing out of the dissolved gases with an indifferent gas, such as nitrogen or carbon dioxide, was shown to be effective enough for use with non-

putrescible liquids but too time-consuming in dealing with activated sludge. Other methods have appeared to be impractical for field use.

Vacuum extraction with heat was used by Adeney,[6] in experiments with sewage, and a modification of Adeney's apparatus has recently been proposed by Damany [7] for use with boiler waters. This method of extraction is also incorporated in the Van Slyke apparatus [8] for the determination of gases in blood.

A disadvantage of the vacuum extraction procedure is that complete removal of the dissolved gases cannot be accomplished without the application of heat or of other auxiliary methods. It will presently be shown, however, that suitable corrections for the failure to achieve 100 per cent recovery of the gas can readily be applied in cases where a delay in the analysis is inadvisable, as in work with activated sludge.

Absorption and Estimation of Oxygen

It appeared possible in early experiments that a satisfactory indication regarding the dissolved oxygen content of a sludge-sewage mixture might be based on a simple measurement of the total volume of the gas obtained by vacuum extraction after removing carbon dioxide and applying a correction for the known solubility of nitrogen. Under the conditions of the activated sludge process, the samples should be fully saturated with nitrogen. In practice, reasonably accurate results were obtained with a minimum of manipulations. The computations, however, were tedious and the apparatus was probably too fragile for field work. Similar objections may be raised against the use of most forms of micro-gas-analysis apparatus.

With a view to the avoidance of corrections for variations in temperature and barometric pressure, use was made of a method described by Theriault and Butterfield.[9] The gaseous oxygen is first absorbed by vigorous agitation in presence of a suspension of manganous hydroxide. The analysis then follows along lines of the well-known Winkler procedure for dissolved oxygen in the absence of any interfering substances. The results are obtained directly in milligrams without troublesome computations.

The Apparatus

In the design of apparatus embodying the principles of vacuum extraction followed by a manganimetric determination of oxygen, it has appeared possible to limit the volume of the sample to 100 ml. Strict accuracy is accordingly sacrificed in favor of portability and convenience in operation. The maximum amount of oxygen available for a test will be 1.0 ml. when samples in equilibrium with air at 0° C. are examined. In equilibrium at any temperature, the volume of dis-

solved nitrogen is approximately twice that of the dissolved oxygen. In work with aerated samples over 1 ml. of gas should be obtained by complete extraction even though dissolved oxygen is practically absent.

Omitting intermediate steps in its development, the apparatus finally adopted is shown in Fig. 1. The evacuation vessel (*A*) consists of a 300 ml. aspirator bottle with an outlet (*B*) near the bottom connected by 4 feet of rubber tubing to a second aspirator bottle of 500 ml. capacity which serves as a leveling bottle. The gas analysis apparatus

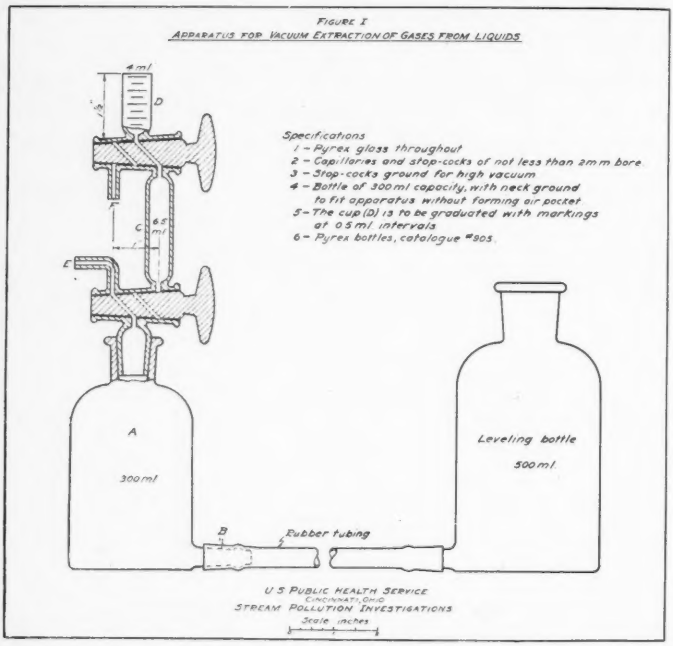

4 ml
D
C
6.5 ml
E
A
300 ml
B
Rubber tubing
Leveling bottle
500 ml
Specifications
1 – Pyrex glass throughout
2 – Capillaries and stop-cocks of not less than 2 mm bore
3 – Stop-cocks ground for high vacuum
4 – Bottle of 300 ml capacity, with neck ground to fit apparatus without forming air pocket
5 – The cup (D) is to be graduated with markings at 0.5 ml intervals
6 – Pyrex bottles, catalogue #905
U.S. Public Health Service
Cincinnati, Ohio
Stream Pollution Investigations
Scale, inches

FIGURE 1
APPARATUS FOR VACUUM EXTRACTION OF GASES FROM LIQUIDS

consists essentially of a 6.5 ml. chamber (*C*) closed at both ends by parallel-bore stopcocks. The upper part of this apparatus is surmounted by a small graduated cup (*D*) and the lower part is ground to fit the evacuation vessel (*A*). Pyrex glass is used throughout. Rubber tubing of the nitrometer variety, size $\frac{3}{16}$ by $\frac{1}{8}$ inch, has proven very satisfactory.

Sampling

For the purpose of minimizing the error due to the loss of dissolved oxygen during sampling, it should generally be advisable to bring the apparatus to the side of a plant rather than to transport the sample to

a laboratory. When air bubbles are absent, as in clarification or settling tanks, a direct connection with glass and rubber tubing should be made between the evacuation vessel and the mixture under examination. This simplification should also be considered in the examination of samples drawn from the lower portions of mechanically aerated tanks.

When air bubbles are present, as in aeration tanks equipped with diffuser tubes or plates, provision must be made for the dissipation of entrained air prior to the removal of the dissolved gases. No allowance need be made for the re-aeration of the sample during collection, the presumption being that an equivalent result should be obtained by sampling a few feet further towards the outlet of the tank. Under these conditions, a wide-mouthed bottle of 125 ml. capacity with an outlet tube near the bottom has appeared to be the most practical type of sampling vessel. The outlet tube is closed with rubber tubing and a pinchcock and the bottle is suitably mounted on a rod. After dipping at the desired location and depth, the rubber tubing is flushed and a connection is quickly made to the sludge inlet tube (*E*) of the evacuation apparatus. With proper allowance for the disappearance of air bubbles, this operation should be completed in 15 seconds.

The Manipulations

Prior to a test, the stopcocks and ground glass connection are carefully lubricated. Approximately 400 ml. of mercury should be present in the aspirator bottles, together with 2 or 3 ml. of water above the mercury in bottle (*A*). With the sludge inlet tube (*E*) open to the air, the evacuation vessel (*A*) is then completely filled with mercury by raising the second aspirator bottle to a predetermined level. When the sludge inlet tube is filled with water, the lower stopcock is turned and the mercury is allowed to fill the gas chamber (*C*) until the upper stopcock is reached. The bore of the upper stopcock should be left full of water.

In testing for leakage the upper stopcock is closed and the leveling bottle is lowered about 30 inches below (*A*) so as to create a Torricellean vacuum. In the absence of leakage, the mercury should again fill all of the evacuated space when the leveling bottle is raised to its original position. As a rule, however, a slight air bubble will be obtained on the first trial, owing to the extraction of gas from the water which covers the mercury. The operation is then repeated. If leakage exists, it may be localized by separate tests of the gas chamber (*C*) and of the evacuation vessel (*A*).

In these manipulations it is advisable to check the upward rush of the mercury by pinching the rubber tubing. It is also important that the lower stopcock be kept open when the upper stopcock is closed.

Otherwise a closed system is created and the gas chamber will be ruptured by any expansion of the mercury. For convenience and safety in handling, the leveling bottle should be kept in a tray suitably equipped with handles.

For a test approximately 100 ml. of sample is injected into the evacuated apparatus through the inlet tube (*E*). The volume of sample admitted to the evacuation vessel may be judged by markings on the sample bottle or a more accurate measurement may be based on the liquid remaining after evacuation.

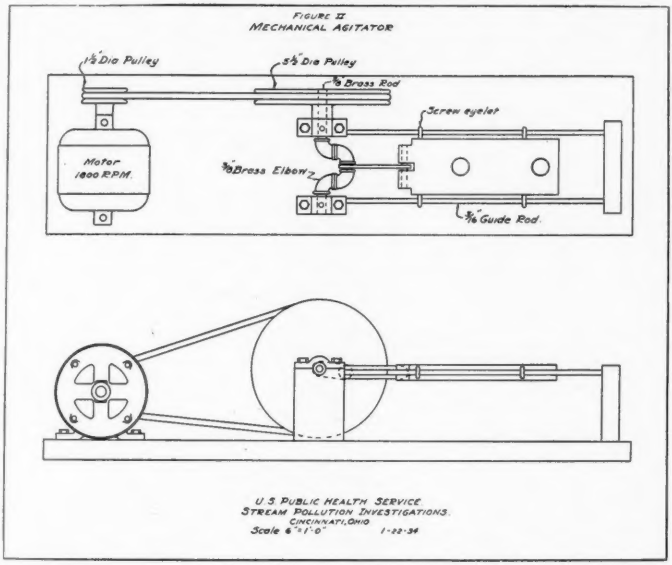
In tests with pure liquids the extraction may profitably be continued for about 2 minutes, the liberation of the gases being facilitated by the gentle rotation of the evacuation vessel so as to stir the sample. With activated sludge, however, it will be advisable to extract only a fraction of the total gas, as described below. After the extraction has proceeded to the desired degree of completion, the leveling bottle is raised so as to transfer the gases to the absorption chamber (*C*). The lower stopcock is closed when the liquid portion of the sample is about to enter the gas chamber. In field work, the determination may be interrupted at this stage and the analysis may be completed under laboratory conditions.

For the absorption of the extracted oxygen 1 ml., or thereabouts, of the usual manganous sulphate solution (480 grams of $MnSO_4 \cdot 4H_2O$ per liter) is placed in the graduated cup (*D*) and 0.5 ml. of this solution is admitted to the gas chamber (*C*) by cautiously turning the upper stopcock. The excess of reagent is voided through the outlet tube (*F*) and the cup is rinsed to remove any adhering solution. The alkaline-iodide solution (500 grams of NaOH and 150 grams of KI per liter) is then introduced in similar manner, again adding only 0.5 ml. and wasting the excess of reagent. The vacuum which still exists in the gas chamber should then be broken by admitting 3 or 4 ml. of distilled water of known dissolved oxygen content.

The gas analysis apparatus is next detached from the evacuation vessel and it is shaken vigorously until the absorption of the oxygen by the manganous hydroxide is complete. Manual agitation is not practical. At least 10 minutes should be allowed for the absorption even with a mechanical agitator capable of 400 to 500 alternations per minute. A large sputum shaker of the friction-drive type has proven very satisfactory. An easily constructed agitator is shown in Fig. 2.

When the absorption of the oxygen is complete, the gas analysis apparatus is again placed in position above the evacuation vessel (*A*) and the precipitated manganese hydroxides are dissolved by introducing approximately 1 ml. of 1:1 sulphuric acid through the cup (*D*). It will generally be necessary to assist the entry of the acid into the

adsorption chamber by the alternate application of suction or pressure to the cup (*D*) with a rubber bulb. The appearance of a reddish coloration in the acid when the upper stopcock is opened is probably due to the desiccation of manganese salts and not to the decomposition of iodides by strong acid. This coloration disappears on dilution and it does not give a blue color with starch solution. The admission of the acid to the absorption chamber may also be facilitated by the previous removal of the residual gas with suction, so as to create a vacuum in the gas chamber (*C*). This may be done by connecting the cup (*D*) and the inlet tube (*E*) with rubber tubing and creating a vacuum in



the evacuation vessel (*A*). The residual gas may then be removed by suitable manipulation of the stopcocks.

The liberated iodine is then transferred to a titration vessel, adding rinsings and distilled water to bring the volume of the solution to about 100 ml. The titration is completed with thiosulphate solution and starch in the usual manner, using a 5 or 10 ml. burette.

Most of the purely analytical sources of error in the Winkler method are avoided by working only with the gas. Nitrites will interfere only if acid, as from cleaning operations, is present above the mercury when the sample is introduced. Under these conditions carbon dioxide may also be liberated in amounts beyond the capacity of the 6.5 ml. chamber. The iodine liberated in the final stage of the process should not

come into contact with mercury which may be held in the bore of the stopcocks by excessive amounts of lubricant. Pipe-stem cleaners are convenient in avoiding this difficulty.

Calculations

Starting with 100 ml. of sample, the calculations are as follows:

A = ml. of 0.025 N thiosulphate solution required = 0.75
B = Total milligrams of oxygen = 0.2 A = 0.150
C = Correction for distilled water = $\frac{4 \times 7.50}{1000}$ = .030
D = Milligrams of oxygen extracted from 100 ml. of sample = .120
E = Apparent oxygen content (milligrams per liter) = 10D = 1.20

The correction of .030 milligrams is based on the assumption that the vacuum was broken with 4 ml. of distilled water containing 7.50 p.p.m. of dissolved oxygen. The possibility of applying a correction for the failure to achieve 100 per cent recovery of the dissolved oxygen will presently be discussed.

Precision of the Method

It is suggested that the precision attainable with a given piece of apparatus be determined by preliminary experiments with distilled water of known oxygen content, using different periods of evacuation and of absorption. It can then be readily shown that approximately 97 per cent of the dissolved oxygen is removed when the sample is stirred under vacuum for 2 minutes, provided that an efficient system of mechanical agitation is employed for the subsequent absorption of the extracted gas. The discrepancy is due to the partial pressure of the water vapor which, at ordinary temperatures, will account for 2 or 3 per cent of the total pressure. A correction for this type of error can readily be applied in dealing with non-putrescible liquids.

Having demonstrated the efficiency of the extraction and, incidentally, the adequacy of the mechanical agitation, tests should next be made to determine the percentage of recovery effected when the period of evacuation is necessarily reduced to a minimum, as in work with stale sewage or activated sludge. With the apparatus at hand, approximately 60 per cent of the total dissolved oxygen is removed during the 15 seconds which are required for the injection of 100 ml. of distilled water into the evacuated space plus the time devoted to 5 or 6 rapid rotations of the evacuation vessel before transferring the extracted gas to the absorption chamber.

With 97 per cent recovery, an apparent oxygen content of 1.20 p.p.m. might accordingly be corrected to read $1.20/0.97 = 1.24$ p.p.m., although a correction of this magnitude will usually be negligible in sewage work. With a percentage recovery of only 60 per cent, the corrected value becomes $1.20/0.60 = 2.00$ p.p.m., again assuming that the apparent oxygen content was 1.20 p.p.m. A careful examination of various possible sources of error has indicated that any discrepancy introduced by this calculation will be well within the tolerances in sewage work. Repeated tests have shown that the percentage recovery under controlled conditions is dependably constant within a variation of about 5 per cent. Using partly deaerated water, it can also be shown that the percentage recovery is sensibly the same whether 2, 4 or 8 p.p.m. of dissolved oxygen are present.

From the foregoing considerations it may be considered that the allowable error in tests for dissolved oxygen by the proposed procedure should not exceed 10 per cent. For the purpose at hand, this degree of precision has appeared to be entirely satisfactory. Following the usual technique of working only on the supernatant liquor from the settled sludge, negative results have generally been obtained whenever the true dissolved oxygen content was 2.0 p.p.m. or less. The error in this case is absolute and it is not subject even to empirical correction.

Suggested Applications

Attempts at reducing operating costs through the avoidance of wasteful amounts of air, or through its more efficient distribution, should evidently be based on accurate knowledge of what constitute an adequate supply. Otherwise, the efficiency of the procedure may be impaired through false economy. Theories of "bulking" based on under-aeration (or over-aeration, for that matter) can never be resolved without accurate information regarding dissolved oxygen values. Systematic studies of the air requirements of the activated sludge process, now in progress at this laboratory, have already yielded some highly interesting information and have furnished promising results. An accurate method for the determination of dissolved oxygen is likewise a pre-requisite to the rational attack of other plant problems, such as the evaluation of the net usefulness of reaeration tanks, the efficient placement of air tubes or plates, the localization of anaerobic conditions, the rating of aeration devices, etc.

From a different angle it has appeared that the apparatus described in this paper might be adapted to the determination of dissolved oxygen in boiler waters and other liquids where a minor degree of reaeration during the collection of the sample may introduce a relatively huge error in the end result. Sampling difficulties should be entirely avoided

by direct connection to the evacuated space. Without increasing the size of the apparatus, it should be possible to secure greater precision by combining the gas obtained from the extraction from several 100 ml. portions.

Acknowledgments

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COMPARISON OF AERATION SYSTEMS FOR THE ACTIVATED SLUDGE PROCESS

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Translated by Willem Rudolfs from 1932 Report of The Dutch Government Institute for Waste Treatment, Holland

"... the rate of solution of oxygen from the air at the exposed surface of the liquors, and the rate and mechanism by which the dissolved oxygen is thence distributed through the unexposed portions of the liquors, are obviously the controlling factors of the efficiency of the (activated sludge) process." Prof. W. E. Adeney, *Jr. of the Royal San. Inst.*, 1932, LIII, 176.

In accordance with the adage "the proof of the pudding is in the eating" it is logical to compare the aeration systems used for the activated sludge process and to perfect each system experimentally by the determination of its purification-capacity. In the yearly report for 1931 * the "purification-capacity" of an activated sludge plant for a given sewage was defined as the number of grams "oxygen demand" which can be satisfied by 1 cu. meter aeration tank per hour, with complete stabilization of the material. In the discussion of the method it was shown that the determination of the purification-capacity of one installation, using one specific sewage with one particular combination of operation conditions, required three months of difficult work. This is principally due to the variations in sewage, especially that coming from cities with many industries and a combined system. No wonder that the purification-capacity of a plant is not often determined. A method generally applicable is greatly needed. Such a method should be simple and at the same time rather exact for the determination of the efficiency of aeration installations. Since aeration is by far the most important function of these installations, it is logical that the aeration capacity or "oxygenation-capacity" is the desired measuring stick.

If an aeration tank is filled with clean water and the water freed chemically of the oxygen present, and then the aeration mechanism started, the oxygen content can be determined at regular intervals by proper sampling. A time-curve of oxygen absorption in different places in the tank can be constructed from the results obtained. This curve, constructed with time as abscissa and oxygen content as ordinate, will have a more or less steep slope towards the oxygen saturation value of the water, which it parallels asymptotically.

* See *Sewage Wks. Jr.*, **5**, 346, 1933.

Adeney and his co-workers[1] investigated with the help of the so-called "narrow-bore-tube and rising-bubble-method" the rate of oxygen absorption from air through a mass of water, which is kept homogeneous by thorough mixing. They found that the rate is: (1) directly proportional to the oxygen deficiency of the water (oxygen saturation value minus oxygen content); (2) directly proportional to the aeration-area (the size of the contact area between a volume-unit of water and air); (3) dependent upon the temperature, according to the following formula:

$$\frac{dc}{dt} = A \times 0.00594(T - 239)(c_s - c_t) g/h/m^3,*$$

where:

$\frac{dc}{dt}$ = rate of O_2 absorption from the air at time t in $g/h/m^3$,
A = aeration-area per volume-unit water in m^2/m^3,
T = absolute temperature,
c_s = saturation-value of the water for air-oxygen at the given temperature and pressure in g/m^3,
c_t = oxygen content of the water at time t in g/m^3.

If two or more points on the above discussed curve are determined for any place in the aeration tank and in addition the saturation-value and the temperature of the water are known, the aeration-area A can be calculated and used as a measure of the aeration capacity.

The cooperation of the Department of Public Works of the city of Hilversum made it possible to make tests in their experimental plant for the demonstration of the above principle.

Adeney's formula was nicely proved by the following test. In a tank with a surface-area of 12.5 square metres and a volume of 46 cu. metres, therefore with a natural aeration-area of 0.272 m^2/m^3, the aeration area was determined at three spots, while the submerged paddles made 7 revolutions per minute and no diffused air was supplied. Near the surface, near the middle and near the bottom of the tank the aeration area A was calculated as respectively 0.280, 0.281 and 0.280 m^2/m^3. It can be seen that these figures are close to that for the natural aeration area.

An idea of the closeness attainable when determining the aeration-area is shown in the results given in Table I. In a tank with a surface area of 18.8 m^2 and a volume of 50 m^3, therefore with a natural aeration

* This is a modified form of the formula given by Adeney l. c. p. 68:

$$W = W_1 - (W_1 - W_0)\, e^{-f(a/v)t}$$

which with the additional symbols:

$$c_t = c_s - (c_s - c_0)\, e^{-f(A/100)t}$$

and where for air: $f = 0.0099(T - 239)$ per min. $= 0.594(T - 239)$ per h.

area of 0.376 m^2/m^3, four tests were made while the artificial surface aeration was kept as constant as possible. The aeration-area A at the three sampling spots is shown in Table I.

TABLE I.

Test	Surface	Middle	Bottom	Test Average
1	4.07	4.13	3.98	4.06
2	4.45	4.38	4.47	4.43
3	4.50	3.98	4.25	4.24
4	4.27	4.11	4.19	4.19
Average at sampling spots	4.32	4.15	4.22	——
Total average	——	——	——	4.23

The maximum variation in the sampling spot averages was 2 per cent, while the test averages vary at most 5 per cent from the total average. The artificial aeration multiplied the natural aeration area $4.23/0.376 = 11.3$ times.

The three curves of oxygen absorption determined by test 4 are shown in Fig. 1.* The fact that the curves are nearly parallel and that the three aeration-areas were found to be nearly alike, shows that none of the three sampling spots were "dead" in relation to the aeration. The lag of the middle and bottom curves compared with the surface, gives an indication of the circulation and the mixing of the tank contents.

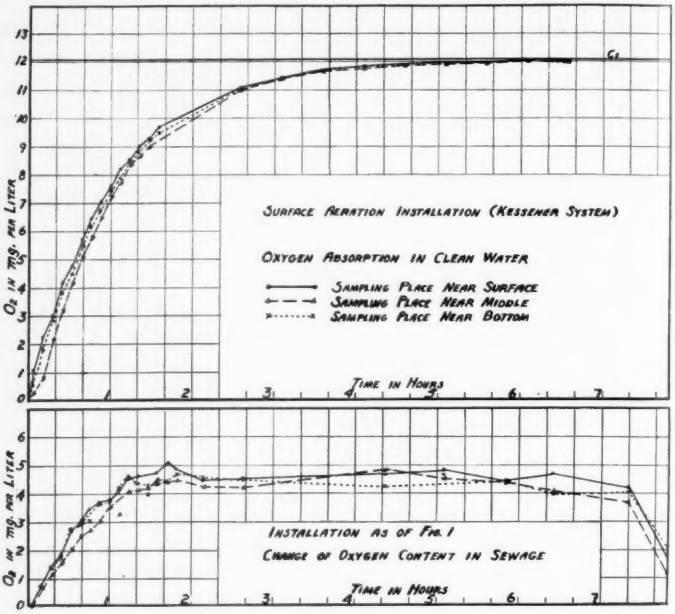
A complication arises in determining the aeration-area in tanks using compressed air. The curve of oxygen absorption does not approach the saturation value of the water at atmospheric pressure, but attains a higher value, due apparently to the air bubbles which are under higher pressure. Since the saturation value is directly proportional to the air pressure, the theoretical pressure during aeration can be calculated from the two experimentally determined saturation values. This pressure appears to exceed the barometric pressure by something less than half the head of water above the diffusers. This is illustrated in Figs. 2 and 3, which show the results of two different compressed air installations with a depth of about 3.5 metres.

For the comparison between a compressed air installation and a surface aeration scheme, the aeration area A is therefore insufficient. Only in the rate of oxygen absorption dc/dt is the effect of A as well as the air pressure expressed. For direct comparison the dc/dt must be con-

* The types of aeration systems used are given in Figs. 1–3. It is obvious that no judgment or comparison of the systems should be based on these preliminary tests, only holding good for one application of each.

verted to the same group of conditions, namely: $T = 283° = 10°$ C.; barometric pressure 760 mm.; distilled water; $c_t = o$.

This defined value, which is of the same character for an aeration installation as the aeration-area A, will be designated as the "*oxygenation capacity*" $(dc/dt)_0$.


Surface Aeration Installation (Kessener System)
Oxygen Absorption in Clean Water
Sampling Place Near Surface
Sampling Place Near Middle
Sampling Place Near Bottom
Time in Hours
O2 in mg. per Liter
Installation as of Fig. 1
Change of Oxygen Content in Sewage
Time in Hours
O2 in mg. per Liter

Figs. 1, 4. Oxygen absorption in a surface aeration installation (brushes, Kessener). Upper: with clean water; lower: with sewage.

For surface-aeration the formula is:

$$\left(\frac{dc}{dt}\right)_0 = A \times 0.00594 \times 44 \times 11.25 = 2.94A$$

and for compressed air:

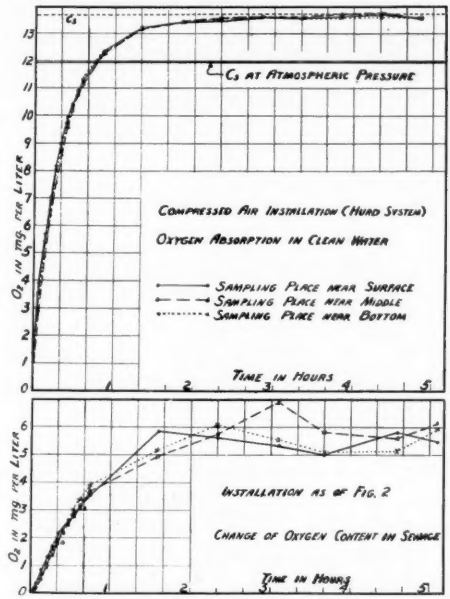
$$\left(\frac{dc}{dt}\right)_0 = 2.94A \times \frac{P_{760}}{760}$$

where P_{760} is the theoretical air-pressure during aeration, calculated for barometric pressure 760. The rate of oxygen supply in the surface aeration installation of Fig. 1 is thus:

$$\left(\frac{dc}{dt}\right)_0 = 2.94 \times 4.23 = 12.4 g/h/m^3.$$

In three trials with a compressed air installation, and constant air supply, the average rates of 3 sampling spots were 49.0, 46.0 and 47.6 $g/h/m^3$ or a deviation of only 3.5 per cent from the total average.

Experimentation with settled sewage instead of with clean water does not show such regular oxygen absorption curves. This is graphically shown in Figs. 4, 5 and 6, with the same installations as used for Figs. 1,


C_s
C_s AT ATMOSPHERIC PRESSURE
COMPRESSED AIR INSTALLATION (HURD SYSTEM)
OXYGEN ABSORPTION IN CLEAN WATER
SAMPLING PLACE NEAR SURFACE
SAMPLING PLACE NEAR MIDDLE
SAMPLING PLACE NEAR BOTTOM
O_2 IN mg. PER LITER
TIME IN HOURS
INSTALLATION AS OF FIG. 2
CHANGE OF OXYGEN CONTENT IN SEWAGE
O_2 IN mg. PER LITER
TIME IN HOURS

FIGS. 2, 5. Oxygen absorption in a direct air installation (porous plates, Hurd). Upper: with clean water; lower: with sewage.

2 and 3. The dissolved oxygen increases more slowly, becomes constant and thereafter decreases. This phenomenon has been reported previously by Parsons and Wilson.[2] From the level of the dissolved oxygen temporarily reached, an idea may be obtained of what the oxygen absorption capacity is under similar conditions. A comparison of the systems now turns out quite differently. The compressed air installation, which gives in clean water a rate of oxygen supply about four times greater than the surface aeration installation, is with sewage hardly any higher (respectively about 6 and 5 p.p.m.). Another compressed air installation, which with clean water was about the same as the surface-aeration installation, gave during a few hours an oxygen level of about 1 p.p.m. after which no free dissolved oxygen could be found. The ac-

tion of the biological oxygen consumption by the sewage changes the picture in all these cases. It is also clear that the sewage influences the oxygen absorption much more detrimentally with compressed air than with surface aeration. This fact can probably be shown more clearly when experiments are repeated with sterilized settled sewage. It may possibly be explained by the sewage colloids covering the air bubbles.

The question may be raised whether the conditions in an aeration tank of the activated sludge process are more similar to those in a tank filled with clean water or to those in a tank filled with settled sewage. An experiment with sterilized and chemically precipitated sewage or with sterilized activated sludge may possibly answer this question.

Before discussing the importance of determinations of oxygenation-capacity in activated sludge research, a brief description of the method of the determinations may be of interest. It should be emphasized that the method is still open to improvement and simplification. Several points will be brought out in the discussion.

In an aeration tank of known volume and surface, clean water is de-oxygenated by stirring a mixture of ferrous sulphate and sodium hydroxide into the water. The amount of ferrous sulphate is so calculated that after a short time a condition is reached whereby the dissolved oxygen of the water has fallen below 1 p.p.m. and no ferrous iron can be detected. At this moment the aeration device is started and samples are taken at regular intervals by means of three siphons placed at different levels in the tank. Thermometers are inserted in the siphons and barometric pressure is recorded. One questionable point is how much the quantities of chemicals used, and especially the ferric hydroxide floc floating in the water, influence the rate of oxygen absorption. To determine the saturation value of the water at atmospheric pressure a vessel with intense surface aeration is used.

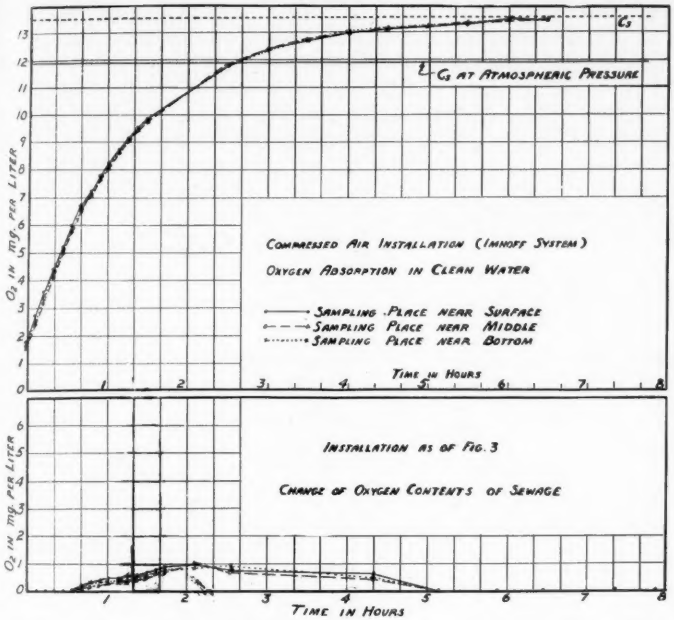
The oxygen dissolved determinations were made according to Winkler, with phosphoric acid instead of HCl; in case nitrites are present sodium azide is added.[3]

Curves of oxygen absorption are shown in Figs. 4–6.

From the formula $\frac{dc}{dt} = A \times 0.00594(T - 239)(c_s - c_t)$ integrated:

$$0.00594(T - 239)A = \frac{1}{t_1 - t_2} \ln \frac{c_s - c_1}{c_s - c_2} = \frac{2.303}{t_1 - t_2} \log \frac{c_s - c_1}{c_s - c_2}.$$

Constructing a second graphical representation with time as abscissa and $\log (c_s - c_o / c_s - c_t)$ as ordinate, the successive sampling results should form a straight line; the tangent of the angle of this line with the abscissa, multiplied by $2.303/0.00594(T - 239)$ gives the aeration-area A (in m^2/m^3). Fig. 7 gives an example, based upon the results obtained


Cs
Cs AT ATMOSPHERIC PRESSURE
COMPRESSED AIR INSTALLATION (IMHOFF SYSTEM)
OXYGEN ABSORPTION IN CLEAN WATER
SAMPLING PLACE NEAR SURFACE
SAMPLING PLACE NEAR MIDDLE
SAMPLING PLACE NEAR BOTTOM
O2 IN mg. PER LITER
TIME IN HOURS
INSTALLATION AS OF FIG. 3
CHANGE OF OXYGEN CONTENTS OF SEWAGE
O2 IN mg. PER LITER
TIME IN HOURS

FIGS 3, 6. Oxygen absorption in a diffused air installation (porous plates and paddles, Imhoff). Upper: with clean water; lower: with sewage.

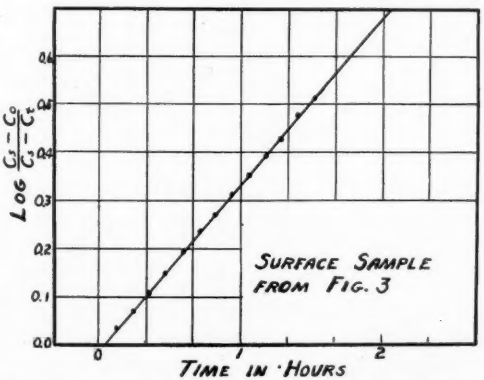


FIG. 7. Example of a graph to be used in calculating the aeration-area.

with the surface sample shown in Fig. 3. In cases where only such slight deviations from the straight line occur as shown in this example, a simplified calculation for A covering a few points could be allowed. In

general, however, it is advisable to draw a straight line through a series of slightly deviating points. As explained before, the calculations of the oxygenation capacity is according to the formula:

$$\left(\frac{dc}{dt}\right)_0 = 2.94A \times \frac{P_{760}}{760} g/h/m^3.$$

The theoretical pressure of the aeration-air at atmospheric pressure 760, P_{760}, must be calculated from the ratio between the saturation values, which are reached respectively in the actual installations and in a separate container with surface aeration.

In the construction of Figs. 4–6 the presence of organic materials has been neglected, since at that time no suitable modification of the Winkler method * was available. The results of these oxygen dissolved determinations are therefore too low; however, the curves can be compared with each other.

Discussion

"... it must be admitted that our knowledge of the (activated sludge) process is far from complete, ... and much fundamental research is necessary before it will be possible to specify the particular type of plant most suitable for the purification of any particular sewage."—Dr. H. T. Calvert, *Jr. Roy. San. Inst.*, XLVII, 295, 1926–27.

The use of aeration capacity determinations and the place which they can assume in activated sludge research is best demonstrated by a discussion of the possibility of *finding the best activated sludge plant for complete purification of a certain sewage.* This can be accomplished by comparing the results of operation of parallel installations of different systems. Each of these plants should allow of great variations in operation. Tests made in this way are costly and time-consuming, especially since after every change in operation, time must be allowed for biological activities to adjust themselves. Moreover, the results obtained cannot be used very well for generalizations, but apply to the individual conditions.

By the introduction of the principle of "oxygenation capacity" the ultimate aim can be reached sooner and in a more systematic way, and the results are of more general interest. The following discussion assumes that there is a definite relationship between oxygenation-capacity and purification-capacity. This is based upon the theory that variation in aerobic organisms (deoxygenation) is one of the governing factors in the activated sludge process. The investigations will fall then into two groups: (1) To determine the relation between oxygenation capacity

* Since then Theriault and McNamee (*Ind. Eng. Chem. Anal. Ed.*, **4**, 159, 1932) have demonstrated that organic material affects the determination especially in the alkaline phase of the Winkler method. Their modified method has many advantages.

and purification capacity, by the use of one local test installation, (2) to determine the relation between the oxygenation-capacity of different aeration systems and total annual cost. The latter investigations do not have to be made at any particular place, but the results must be interpreted with the help of local cost data. Combination of the results of the two groups of investigations makes it possible to deduce, for a given sewage, the relation between purification capacity and costs.

A more or less schematic presentation of the procedure to be followed for an investigation of the best activated sludge installation for complete purification of a given sewage is shown below.

Procedure

I. To find the relation between oxygenation-capacity and purification-capacity for a given sewage

A. Test Unit

The test unit to be built for the treatment of settled sewage should conform to the following requirements:

1. The degree of aeration must be flexible within very wide limits, easy to keep constant, easily defined and reproduced.

2. In order to make it possible that any device and type can be used, the aeration tank should consist of some ten compartments with a settling tank at the end, while a second settling tank can be connected with any compartment. Each of the settling tanks should have provision made for return sludge. Provision should be made to introduce influent as well as returned sludge into any compartment desired.

3. Continuous and, if possible, self-registering measuring instruments should be installed for all factors which influence the purification capacity or power consumption (for influent, return flow, wasting of sludge, air, temperature, sludge percentage, rainfall), and equipment for the determination of purification-performance, power-consumption, sampling.

B. Oxygenation-Capacity Determinations

In one of the compartments of the test unit a series of oxygenation capacity determinations are made which can be well defined and reproduced.

C. Purification-Capacity Determinations

The determinations fall into three groups:

a. Normal Tests.—The installation is operated as "single" unit. The compartments receive the same quantities of air, while the sludge percentage, return flow and temperature are "normal." A curve show-

ing the relation between purification capacity and the results obtained under *B* is plotted. On the assumption that dissimilarity of aerobic organisms is one of the principal factors in the activated sludge process, this curve will show a line of the type *A*, given in Fig. 8.

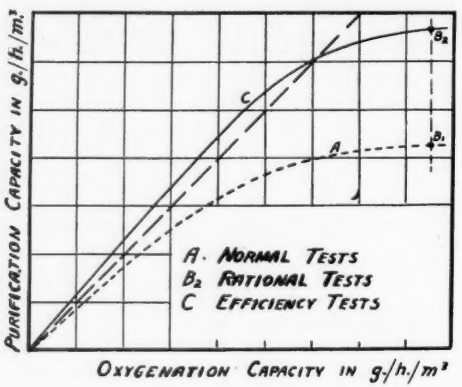


FIG. 8. Relation between oxygenation capacity and purification capacity.

b. Rationalization Tests.—These tests are all performed with *a large oxygen supply capacity*, which by the "normal-tests" has shown a purification capacity b_1, in the nearly horizontal part of the curve. By changes in operation an increased purification capacity (point b_2) is aimed at, keeping in mind the changes in cost of operation. Important changes in operation are:

1. Change of sludge percentage in the aeration tank,
2. Change in return flow,
3. Intermediate discharge of effluent,
4. Pre-treatment of sewage (*e.g.*, neutralization, equalizing, pre-aeration with or without activated sludge),
5. Gradations of aeration,
6. Wasting of surplus sludge in an (adsorptively) loaded condition rather than in an (oxidatively) regenerated one.

The adoption of the last measure is immediately clear: the quantity of oxidation required is less. This is not true for the remaining changes in operation, which affect the rate of purification only in consequence of the betterment of biological and physical-chemical conditions. Our knowledge regarding these conditions is extremely limited, but the disadvantage of insufficient knowledge should not be over-emphasized. The technician knows which operation factors can be changed; after the changes have been made he must wait until nature has taken its course.

Biological and physical-chemical information will however greatly help when a choice must be made. Each test costs time and effort and it depends upon the technician's judgment whether or not the particular study will show an improvement.

A brief (necessarily incomplete) discussion of the relation between the stated combinations and the biological and physical-chemical conditions is justified. The sludge concentration influences the number of microorganisms and determines the size of the adsorption surface of the floc where the biochemical changes take place.* The return flow and the intermediate discharge of purified sewage influence the concentration of organic substances which are already or not yet absorbed. It affects the concentration of substances which might be detrimental to the organisms and of the products produced by them; pre-treatment has the same object. The latter may also affect the relative hydrogen-ion concentration of the water and the isoelectric point of the colloids, considered of great importance by Baly (4). By gradation of aeration it may be possible to balance the oxygen supply with the oxygen demand during the different phases of purification.

Chemical stimulation of the bacteria remains to be investigated. There is one factor, temperature, which is undoubtedly important, but which cannot be controlled in practice. This is unfortunate, because it is impossible to predict how temperature fluctuations may affect the purification capacity. In the *Addendum* a brief discussion is given dealing with this factor. The tests should be made by preference during average yearly temperature conditions.

c. Efficiency Tests.—Suppose it has been possible by the "rationalization-tests" to increase the purification capacity to point b_2 in Fig. 8. The efficiency tests are then further necessary to plot a curve which shows the relation between purification capacity under operation conditions which made it possible to reach b_2. This curve will have the same type as line *C* in Fig. 8.

II. TO FIND THE RELATION BETWEEN OXYGENATION-CAPACITY AND COST AT PREVAILING PRICES

In order to develop the aeration systems in such a way that the oxygenation capacities wanted are obtained at minimum cost, it is desirable to have a test installation for each system, where all factors which affect oxygenation capacity, power and construction costs can be varied and carefully measured. By systematic variation of all factors an attempt

* (This statement is in its generality correct, but the reader should not gain the impression that the numbers of microorganisms (either bacteria, protozoa or fungi) necessarily vary directly with the concentration, or that double the number will do twice the amount of work. Translator.)

is made to perfect the installation, while every time the oxygenation capacity is determined and the yearly cost for a large plant is estimated on the basis of local costs of land, cement, machinery, patent fees, capital, maintenance, energy, labor and management. The aeration capacity determinations indicate whether there are spots in the tanks which are more or less "dead" in relation to aeration, circulation and mixing. By means of special tests it can be determined whether parts of tanks are more or less "dead" in relation to displacement (short-circuiting tests) and whether the sludge remains sufficiently in suspension (settling tests). It is then possible to construct an optimum curve for each system showing the relation between oxygenation capacity and local costs. Fig. 9 shows two curves which might be characteristic of

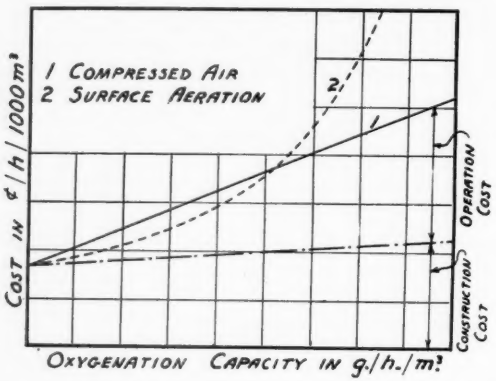


Fig. 9. Relation between oxygenation capacity and cost.

respectively (1) a compressed air system and (2) a surface aeration system.

III. Deduction of the Relation between Purification-Capacity for a Given Sewage and Cost at Prevailing Prices

If in activated sludge installations of different aeration-systems the purification capacity under the same operating conditions depended exclusively upon the oxygenation capacity, it would be possible to deduce the relation between purification capacity and cost directly from the optimum curves presented in Figs. 8 and 9, as shown in Fig. 10. A tangent from the origin to the curves of this figure would show the point indicating the highest ratio (Purification Capacity/Cost). The most economical oxygenation capacity and the aeration system to be used, as well as the required aeration time, would be thereby determined.

Since the assumption at the beginning of this paragraph is incorrect, combination of the results obtained in Sections I and II becomes more complex.

The purification capacity of an installation under certain operation conditions depends not only upon the oxygenation capacity, but also upon the pressure of the aeration-air. For a surface aeration system this pressure is the same as atmospheric pressure. For compressed air systems the deeper the diffusers the higher will be the pressure; this value is known from the oxygenation capacity determinations. The saturation value of the water rises in proportion to the pressure of the aeration-air, while the oxygenation capacity in turn increases in proportion with the saturation value, because this capacity is defined as the

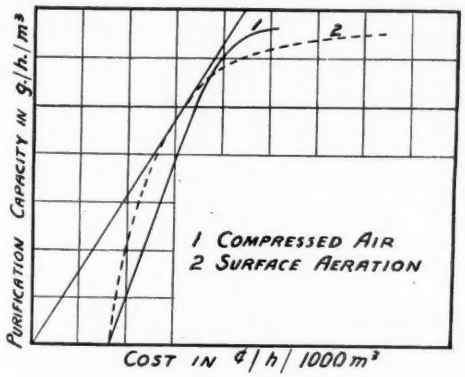


FIG. 10. Relation between purification capacity and cost (from results given in Figs. 8 and 9).

rate of oxygen absorption at a zero oxygen content. The rate of oxygen absorption at any content above zero is dependent upon (in addition to the oxygenation capacity) the ratio between the oxygen deficiency and the saturation value:

$$\frac{dc}{dt} = \left(\frac{dc}{dt}\right)_0 \frac{c_s - c_t}{c_s}$$

This ratio, and consequently the rate of oxygen absorption, increases with the increased value of c_s, or in other words, with the increase in pressure of the aeration-air.

It follows from these statements, that to obtain a certain purification capacity, *i.e.*, a certain rate of oxygen absorption, a larger oxygenation capacity is required in any installation than in a test unit, if its pressure of aeration-air is smaller than in the test unit, and vice-versa. Be-

fore a curve of a compressed air test unit (as in Fig. 8) can be combined with the surface aeration curve (Fig. 9), it is necessary to determine which oxygenation capacities at atmospheric pressure are directly comparable with those used in the test unit. This can be done graphically as shown in Fig. 11, where a curve shows the oxygen contents in the test unit at the different oxygenation capacities. These oxygen contents have been determined during the "efficiency tests" (Fig. 8, Curve *C*). In accordance with this reconstruction of the curves, Fig. 10 is replaced by Fig. 12. The fact that in Fig. 12 (deduced from the totally hypothetical curves in Figs. 8 and 9) one tangent from the origin accidentally touches both curves, means that the choice is open between the

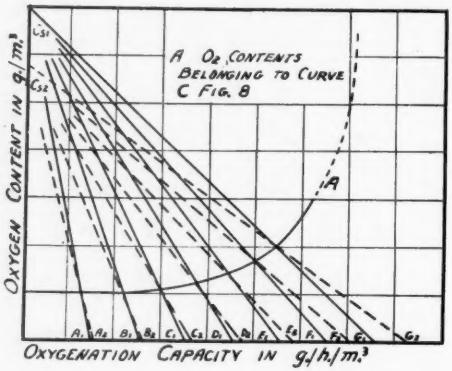


FIG. 11. Interpolation of oxygenation capacities, necessary for definite purification capacities in a compressed air installation (a_1, b_1, etc.) to those in a surface aeration system (a_2, b_2, etc.).

surface aeration system with weaker aeration but longer aeration time, and the compressed air system with stronger aeration and shorter aeration time.

A second complication in the coördination of the results obtained in Sections I and II presents itself if in installations of different aeration-systems the same *oxygenation capacities* (in relation to clean water) are recorded, but different *oxygen absorption capacities* are found with the water-sludge-mixtures. As stated before, this phenomenon has been noticed for strong colloidal sewage (without activated sludge). It seems possible that the adsorption or flocculation of colloids by the activated sludge may be so rapid and complete that the oxygen absorption in the sewage-sludge-mixture is practically not affected by the colloids originally present. Should it be shown by experimentation that there is any measurable effect, it becomes necessary to determine the oxygen

absorption capacities in addition to the oxygenation capacities. For this work testing units of the different aeration systems are necessary at the place where the new plant is to be built.

IV. PRACTICABILITY OF THE INVESTIGATIONS, INDICATED IN SECTIONS I TO III

Detailing the studies sketched in the previous sections shows clearly that a *complete* investigation would require a number of years of intensive work. The fact that only very few projects could bear economically such an investigation is no reason why the whole scheme should be relegated to the domain of fantasy. When the program given under Section I has been worked through once at a given research installation,

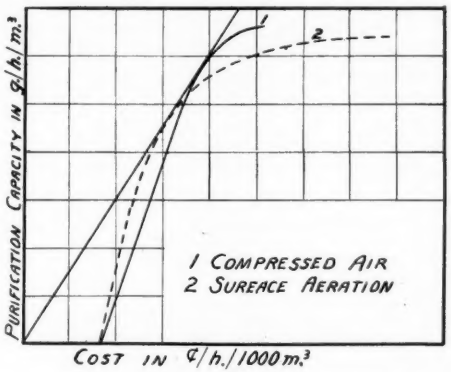


FIG. 12. Relation between purification capacity and cost (after interpolation with help of Fig. 11).

where the sewage is not too abnormal, results will be obtained which will shorten the procedure very materially. The "normal-tests" discussed under Section I C-a are made only to indicate which (rather arbitrary) oxygenation capacity is required for the "rationalization tests." When one curve of Fig. 8 has been established, the required oxygenation capacity for any other sewage can be approximated—after testing the B.O.D. in the laboratory—without "normal-tests." The results of one set of complete "rationalization tests" will undoubtedly be applicable to a large extent to other sewages (sludge wasting, air gradation, pre-treatment with sludge, etc.). In regard to the effect of the other operation factors (return flow, concentration of sludge in aeration tank, etc.), the results will give a sufficient insight to make the choice of tests for other sewages much simpler. The "efficiency tests," which might successively be made with sewage in different stages of primary treatment, are then left as the most important work of Section I.

The work under Section II is mostly done without sewage and can be carried out independently of local conditions. In case no test unit of a certain aeration system is available, a study of oxygenation capacities and cost of different existing activated sludge plants of this particular system can be made, possibly after its improvement. The result will be that a number of aeration systems can undoubtedly be eliminated, while for other systems the necessary improvements to make them economical and applicable for divergent local costs can be indicated.

Addendum

Effect of Temperature upon the Activated Sludge Process

The effect of temperature upon the activated sludge process is many-sided and by no means clear. The importance of this factor for the comparison of different activated sludge systems is sufficiently great to make a careful analysis desirable, in spite of the fact that in doing so a few hypotheses must be used.

The following is therefore intended only as an "outline for a working hypothesis."

The temperature affects in the first place the rate of oxygen absorption:

$$\frac{dc}{dt} = A \times 0.00594(T - 239)(c_s - c_t) g/h/m^3.$$

A rise in temperature causes an increase in the factor $(T\text{-}239)$ and a decrease in the factor $(c_s - c_t)$. The latter factor overbalances the former, as indicated in Fig. 13. Accidentally, in the temperature range given,

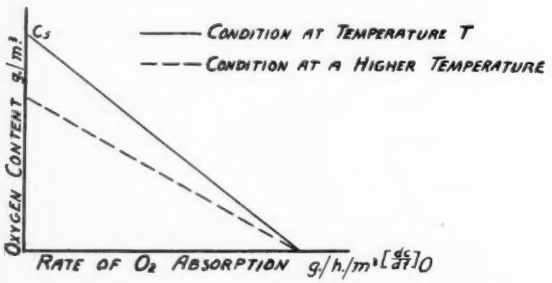


Fig. 13. Effect of temperature upon rate of oxygen absorption.

the two factors nearly neutralize each other in regard to oxygenation capacity.

The dissimilation * power of an installation is as a rule determined by the rate of oxygen absorption and the "physiological oxygen content," which should not be over-taxed if the sludge is to remain healthy. Fig. 14 shows that the effect of increased temperature upon the rate of oxygen

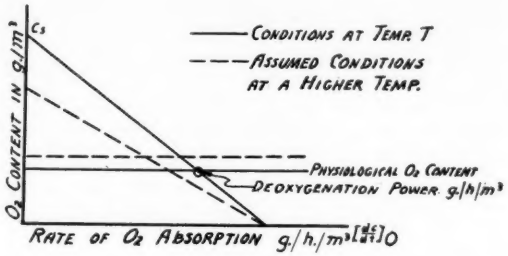


FIG. 14. Effect of temperature upon dissimilation power.

absorption is to lower the dissimilation power (and thereby the purification capacity). Probably the rise in temperature also influences the physiological oxygen quantity itself; a rise in this will result in a further decrease in the purification capacity.

In the third place the temperature influences the rate of (biological) oxygen depletion (dissimilation); in general this will increase when the temperature rises. If the oxygen depletion in the activated sludge process follows Phelps' law,[5] as it does without activated sludge in a river or in a bottle in the laboratory, the effect of temperature can be shown as in Fig. 15. (Note: Phelps' law expresses the rate of deoxygenation as

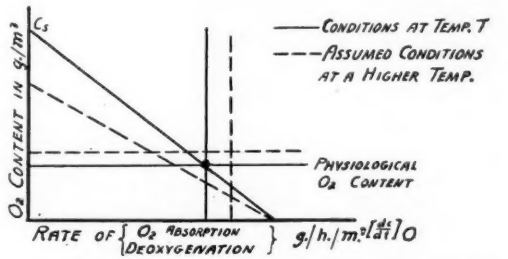

C_s
CONDITIONS AT TEMP. T
ASSUMED CONDITIONS AT A HIGHER TEMP.
O_2 CONTENT IN g./m.³
PHYSIOLOGICAL O_2 CONTENT
RATE OF { O_2 ABSORPTION / DEOXYGENATION } g./h./m.³ $\left[\frac{ds}{dt}\right]O$

FIG. 15. Effect of temperature upon rate of deoxygenation.

directly proportional to the biochemical oxygen demand, but unaffected by the oxygen content.)

* By "dissimilation power" is meant the identical rates of biochemical oxygen depletion and oxygen absorption in an installation receiving its purification capacity load.

Suppose the aeration were increased to such an extent that the influence of a rise in temperature upon the rate of oxygen absorption and upon the physiological oxygen content were compensated, an increased rate of deoxygenation would still cause a decrease in the oxygen content below the physiological limit, which could be prevented by decreasing the amount of organic material. The increase in the rate of deoxygenation will therefore also cause a decrease in the purification capacity.

An exception to this procedure is found in installations where such strong aeration is practiced that the dissimilation power is not determined by the physiological oxygen quantity, but controlled by the maximum rate of deoxygenation which is possible under the given biological conditions. The increase in this maximum rate caused by rises in temperature will result in an increase in the purification capacity as shown in Fig. 16.

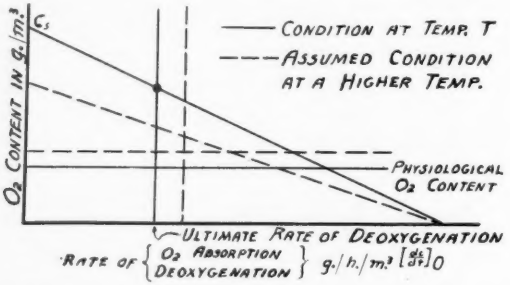

C_s
CONDITION AT TEMP. T
ASSUMED CONDITION AT A HIGHER TEMP.
O_2 CONTENT IN g./m.³
PHYSIOLOGICAL O_2 CONTENT
ULTIMATE RATE OF DEOXYGENATION
RATE OF { O_2 ABSORPTION / DEOXYGENATION } g./h./m.³ $[\frac{dt}{dt}]_0$

FIG. 16. Effect of temperature upon maximum rate of deoxygenation.

The importance of the effect of temperature upon the rate of oxygen absorption (namely upon the saturation value), and upon the rate of deoxygenation is considerable; this is shown graphically in Figs. 17 and 18, which have been taken from Winkler [6] and the U. S. Public Health Reports.[7]

(The authors have appended a brief English summary which is hereby given. Translator.)

SUMMARY

As the determination of the purification capacity of an activated sludge plant with regard to a given sewage (discussed in the Annual Report, 1931) involves great difficulties, it is desirable to procure a simpler method to compare and to improve aeration devices for activated sludge plants.

The oxygenation capacity of an aeration plant—aeration being maintained at a fixed intensity—will be understood to be the rate of oxygena-

FIG. 17. Oxygen saturation values of distilled water at different temperatures and atmospheric pressure at 760 mm.

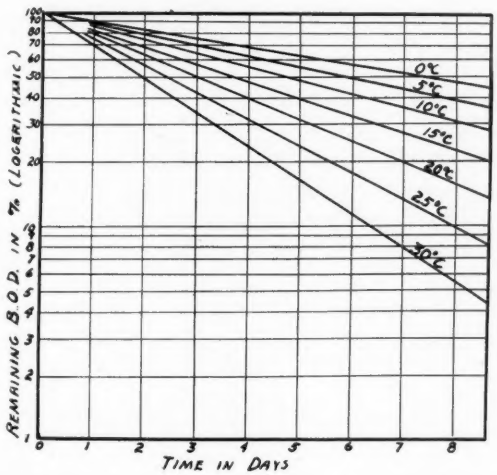


FIG. 18. Reduction in B.O.D. of domestic sewage under aerobic conditions at different temperatures.

tion of fully deoxygenated distilled water in that plant, at 10° C. and 760 mm. barometric pressure; it is expressed by the unit $g/h/m^3$ (grams of oxygen per hour per cubic metre tank volume).

If in an aeration tank a series of points on the oxygenation time-curve for de-oxygenated tapwater are determined, as well as its temperature and the saturation-value towards which this curve tends, then it is possible, thanks to the research-work of Adeney and his collabo-

rators, to calculate the aeration-area (m^2/m^3) and the oxygenation capacity.

A series of experiments on this principle confirmed the formula given by Adeney and demonstrated that the oxygenation capacity of working scale plants can be estimated with a fair degree of precision. Spots more or less "dead" with regard to aeration can be pointed out; circulation and mixing can be inspected.

The oxygen saturation-value of water in an airblowing plant is higher than that in a surface-aeration plant; the theoretical air-pressure appears to exceed the barometric pressure by somewhat less than half the head of water above the diffusers.

Diffused air aeration is more unfavorably influenced by colloidal organic matter in sewage than surface aeration. This fact must still be confirmed by a series of aeration experiments with sterilised sewage. The question whether the conditions of the sewage-sludge-mixture in an activated sludge plant resemble more closely those when using pure water than those with colloidally polluted water must still be solved experimentally.

The value of estimations of the oxygenation capacity in activated sludge research is demonstrated in a discussion on the problem of finding the most suitable and efficient type of activated sludge plant for the complete purification of a given sewage. This discussion is based on the supposition that oxygenation capacity and purification capacity are intimately related, in accordance with the theory that the dissimilation of aerobic microbes is one of the governing factors in the activated sludge process.

The object of a first group of experiments is to trace the relation between the oxygenation capacity and the purification capacity for the sewage in question. For this purpose an experimental plant must be constructed.

In a second group of experiments the relation between oxygenation capacity and costs (cost of construction, energy, etc.) is investigated for the different systems of aeration. These experiments can be carried out everywhere; the results are transferable. Coordination of the results of these two groups of experiments shows the relation between purification-capacity and costs (both dependent on local conditions), indicating which plant will show the most favorable ratio between these two values.

If this scheme of experiments has once been carried out, our knowledge of the activated sludge process will experience considerable advancement.

Finally the influence of temperature on the activated sludge process is discussed.

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FILTRATION AND INCINERATION AS A MEANS OF SLUDGE DISPOSAL *

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As is well known, filtration of sludge is not a new process, since chemically precipitated sludge has been dewatered in this manner for many years. The use of this process spread rapidly throughout England, after about 1880, and a little later became popular in this country. One of the first plants of this type was established at East Orange, New Jersey [1] in 1888. Other plants followed, those at Worcester and Providence being the most important. Although treatment of this kind was in use for several years, chemical precipitation gave way to plain sedimentation and the filtering of sludge was replaced by drying on sand beds.

The digestion of sludge, with subsequent drying on sand beds, has many disadvantages. The anaerobic composition of sludge—especially mixtures of fresh and activated—produces a concentrated supernatant liquor, the disposal of which is a difficult problem. When this supernatant liquor is discharged into preliminary settling tanks, it increases the biochemical oxygen demand as well as the colloidal matter and tends to upset the aerobic biological processes. The dewatering of the digested sludge has caused the most trouble. In case of poor digestion, nuisances from odor and flies result. Rainy or cold weather delays the drying The removal of sludge from sand beds is expensive. The use of glass-covered beds, storage tanks and sludge removal equipment has partially solved these problems but the installation cost of these things is high The cost of land for treatment plants of large cities may be excessive.

The development of the activated sludge process again introduced filtration of sludge. Activated sludge does not dry readily on sand beds and is not satisfactorily disposed of in lagoons or trenches. Vacuum filtration was tried by many cities and adopted by a few, among them Milwaukee, Pasadena and Charlotte, where some success was attained in the disposal of sludge as fertilizer. The process, suggested by Imhoff [2] in 1925, of digesting activated sludge along with fresh solids, soon became popular. The great difficulty with this procedure is the excessive moisture of the sludge. Fully digested sludge (mixture 1:1 ratio of dry solids) containing about 92 to 96 per cent moisture may be so jelly-like in consistency that it will hardly flow.

* Presented before North Carolina Sewage Works Association, Nov. 14, 1933.

These troubles have brought about a return to sludge filtration. Los Angeles [3] started experiments in dewatering various raw and digested sludges, by this means, in 1930. Baltimore, Chicago, Columbus, North Toronto and others soon followed.

Sludge Conditioners

Better and cheaper sludge conditioning has made filtration practicable. After a great deal of experimentation, most plants have adopted the use of ferric chloride as the best of the conditioners, especially since the price has been materially reduced. Although in 1926 the cost of ferric chloride, dry basis, was $180.00 to $200.00 [4] per ton, Chicago and Milwaukee are now paying about $40.00,[5] delivered. Pasadena is using ferric chloride with diatomaceous earth and Charlotte, ferric sulphate.

Paper pulp has also been used to a considerable extent as a sludge conditioner. The York Township (Ontario) [6] plant has been using paper pulp as a filter aid both with and without lime and the plant at Hagerstown, Maryland, paper pulp and lime for conditioning a mixture of activated sludge, fresh solids and fine screenings. The Rockville Center, Long Island plant used paper pulp with and without lime to filter raw sewage and activated sludge effluent and for the dewatering of sludge. Paper pulp makes a very cheap conditioner when it can be reclaimed by the refuse disposal department of a city, as at Rockville Center.

The alkaline filter aids do not seem to give as great a yield of filter cake per unit area of filter as do the acid, but the alkaline aids produce a sludge cake which is not as apt to cause nuisance when disposal on the ground is necessary. This is of the greatest importance in the filtration of fresh solids or mixtures of fresh solids and activated sludge. The alkaline conditioners are not corrosive, as are the acid ones, such as ferric chloride.

Dewatering Fresh Solids

Although Winston-Salem and Dearborn have been dewatering chemically precipitated sludge on vacuum filters for some time, it is only within the last two or three years that work has been done on plain settled fresh solids. After a few tests at Baltimore [7] it was found impracticable to dewater their raw sludge, using ferric chloride as a conditioner. The filter rates were low, the effluent contained a large amount of suspended solids and the filter cloths clogged rapidly, making continuous operation impossible. Slightly better results were obtained using paper pulp, lime and ferrous sulphate but the large amount of conditioner necessary made the cost much more than for digested sludge. The age of

their raw solids, however, was unknown, the pH was very low and it may be that better results would have been obtained had the solids been really fresh.

Goudey,[3] on the other hand, found raw sludge easier to condition and filter than digested sludge. He was able to obtain cake containing 65 per cent moisture without the use of admixtures, heating or return of dry cake. The raw sludge cake was thick and granular except for the normal amount of fibers. It had no objectionable odor and dried in the air in about 24 hours. Filter rates were high and the cloth remained clean. He estimated the total fixed and operating costs for dewatering solids to dryness, at about $6.50 per million gallons of sewage treated. This was about the same as the cost of dewatering and drying digested sludge.

Preliminary work at Chicago [8] indicated that fresh sludge filtered at higher rates and with less coagulant than was required for digested sludge.

At Dearborn,[9] chemically precipitated sludge is dewatered to 60–70 per cent moisture at the rate of 6–8 pounds per square foot of filtering surface per hour. Due to the excess lime, it is reported this cake can be air dried without odor nuisance.

Dewatering Mixed Fresh and Activated Sludge

The Sanitary District of Chicago has operated a pilot plant with a capacity of twenty tons of dry solids per day, which, on a vacuum filter, has dewatered mixtures of fresh solids and activated sludge in a ratio of approximately 1 : 1 by weight of solids. Using 4 to 6 pounds of ferric chloride, dry basis, per 100 pounds of dry sludge, a filter cake containing about 80 per cent moisture has been obtained at rates of 3 to 6 pounds of dry solids per square foot per hour. Filtration of the mixed sludge was preferable to filtration of activated sludge alone, on account of the lower moisture content of the feed, which varied from 95 to 97 per cent. Filtration rates for activated sludge at Pasadena and Milwaukee are 1 to 2 pounds per square foot per hour.

Since 25 to 35 per cent [10] of the iron added as ferric chloride is found in the filtrate during the filtration of mixed sludge, the filter effluent could profitably be returned to the aeration tanks. The iron floc formed would tend to give a more rapidly settling sludge and might help to eliminate bulking. This treatment would also remove the iron from the liquor before its discharge from the plant.

Dewatering Digested Sludge

Dissatisfaction with drying on sand beds has concentrated a great deal of attention on the vacuum filtration of digested sludge. On small

scale operation, Baltimore [7] has obtained rates as high as 27.4 pounds of dry cake per square foot of filter area per hour and it is estimated that with 8 pounds of ferric chloride per 100 pounds of dry solids, a filtering rate of 14 pounds per square foot per hour can be maintained. The sludge was mostly of domestic sewage origin and contained from 4.8 to 9.9 per cent dry solids, of which 51.5 to 61.1 per cent were volatile. The cake contained from 71.9 to 80.5 per cent moisture. The effluent from the filter was of such a character that it could be added to the plant effluent without producing any detrimental effects. The estimated cost of dewatering by vacuum filtration was about $5.00 per ton of dry solids. The average cost of air drying sludge from 1925 to 1932, $6.32 per ton of dry solids, is higher than in many other places because the sludge which is insufficiently digested takes longer to dry, and because it is removed from the beds while it still has a high moisture content.

Goudey [3] reports that in experimental work at Los Angeles digested sludge containing 90 to 94 per cent moisture was easily reduced to 82 to 84 per cent. By adding dry sludge to the filter the moisture could have been further reduced to 55 to 65 per cent. He calculated the cost of chemicals used in dewatering the sludge to 65 per cent moisture at about $3.50 per million gallons of sewage treated and considered vacuum filtration cheaper than air drying.

Flynn [11] states that an investigation of sand bed operating costs including interest and maintenance charges, has shown actual average costs of well managed large plants to be $4.94 per ton of dry solids handled.

At Columbus [12] filter cake containing 70 to 72 per cent moisture was produced from digested sludge at rates of 4 to 5 pounds per square foot per hour.

Incineration

Filter cake from activated sludge and fresh solids, unless heavily dosed with chemicals, produces offensive odors and must be treated further before it can be stored. Considerable progress has been made in incineration methods for the final disposal of sludge. The advantage of incineration is that it destroys organic matter, leaving an inert ash which is entirely free from odors and suitable for dumping.

The B.t.u. value of sludge varies according to its type and source. On an average,[13] fresh solids have a B.t.u. value of about 6500 to 8500, activated sludge 6000 to 8000 and digested sludge 4800 to 7200.

At Chicago, after filtration, the sludge moisture was reduced to about 20 per cent in a rotary drier using the heat from the combustion furnace. The dried sludge was incinerated to furnish the heat for the drier. A small amount of coal was occasionally needed. The odors from the in-

cinerator could be eliminated by raising the temperature to about 1500° F. As a result of the tests, Chicago expects to build a large plant at the West Side for filtering and incinerating mixed fresh solids and activated sludge.

It is a question whether it is cheaper to digest the raw solids and filter the digested sludge or to filter the raw sludge immediately. Certainly, filtration of the raw solids eliminates the cost of digestion tanks and where incineration is to be employed as a final means of sludge disposal, less coal or other fuel must be added to promote burning of the fresh solids than would be needed for digested sludge, since the fresh solids have a considerably higher heat value.

The odors from digested sludge filter cake would probably not be troublesome but it is questionable whether dewatering to 70 to 80 per cent moisture is sufficient unless the sludge is used locally for fertilizer. Partial digestion to provide a balancing reservoir which may be satisfactory from a filtering standpoint would be too expensive unless the tanks were already available.

Conclusions

Tests carried out in various parts of the country indicate that vacuum filtration of sludge, either fresh, activated or digested, is practical and economical for sizable plants. This process can be confined to a small space, whereas with air drying on sand beds, large areas of land are needed, land which may be expensive and difficult to obtain. Filtration eliminates a great deal of labor or expensive equipment for sludge removal. In the case of filtration of fresh solids the cost of digestion tanks is eliminated. This method removes the possibility of offensive odors obtained during the air drying of only partially digested sludge, a process which may be necessary from time to time. Dewatering in this way is entirely independent of weather conditions and the whole process is continuous. In the North, sludge drying on sand beds is unsatisfactory during three or four months of the year and storage tanks or glass covered beds must be provided. If storage tanks or glass covered beds are not included, the cost of vacuum filtration and of drying on sand beds is about the same, the cost of filtration being estimated at $5.00 per ton and of drying on sand beds, $4.94 per ton. If storage tanks or glass covered beds are used, the cost of air drying is higher.

Incineration seems to offer a satisfactory method for the final disposal of sludge especially in the case of fresh or activated sludge where odor trouble with the filtered cake may be encountered.

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RESULTS OF SIX MONTHS' OPERATION OF CHEMICAL SEWAGE PURIFICATION PLANT*

CHICAGO SANITARY DISTRICT

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About a year ago, we presented at a meeting of this Association a paper, in which was described the operations carried out and the results obtained in treating sewage from the Dyckman Street outfall in New York City. We had just concluded arrangements with The Sanitary District of Chicago to build and supervise the operation of a plant of 25,000 to 30,000 gallons per day capacity at the North Side Sewage Treatment Works.

This paper describes the operations and sets forth the results obtained in the operation of the Chicago plant, together with costs of construction and operation covering a series of plants of various sizes. (*Editor's Note:* This paper and the estimates have been prepared by the staff of Guggenheim Bros., and not as a joint report of Guggenheim Bros. and the Sanitary District of Chicago.)

Character of Influent Sewage

The North Side Sewage Treatment Plant treats the sewage originating in an area of about 60 square miles, including the portion of the City of Chicago lying north of Fullerton Avenue, the City of Evanston, and the Villages of Kenilworth, Winnetka, Glencoe, Niles, Niles Center and Tessville. The tributary population is about 1,000,000. There are few, if any, manufacturing plants in the section and, since the water consumption per capita is high, namely, about 300 gallons per person per day, the sewage is of domestic type and rather weak. The time in the sewers before reaching the treatment works varies from 4 to 6 hours. The average temperature of the sewage is about 65° F. During the time of the run, the temperature varied from 53° F. to 79° F. The sewage was of the following character:

* Presented before the Sixth Annual Meeting of the New York State Sewage Works Association, New York City, January 16, 1934.

	Average	Monthly Variation
B. O. D. (5-day)	103.0 p.p.m.	94.0 to 123.0 p.p.m.
Suspended Solids	133.0 p.p.m.	119.0 to 151.0 p.p.m.
Total Nitrogen	16.1 p.p.m.	13.2 to 17.7 p.p.m.
Ammonia Nitrogen	7.4 p.p.m.	5.5 to 8.6 p.p.m.
Organic Nitrogen	8.7 p.p.m.	7.7 to 9.5 p.p.m.
Organic Carbon	83.0 p.p.m.	66.0 to 107.0 p.p.m.
pH	7.2	7.1 to 7.4
Bacteria (13 counts) Total	850,000 col./c.c.	
B. Coli	100,000 col./c.c.	
Hardness	165.0 p.p.m.	

Treatment

The treatment of the sewage is the same as that described in our paper read about a year ago. (See This Journal, Vol. 5, p. 61, Jan., 1933.) As shown in detail in that paper, the process is simple and direct, the sewage being subjected to but two operations:

First: Removal of the suspended matter and the major portion of the non-basic dissolved material by coagulation and precipitation with an iron salt and an alkali, preferably ferric sulphate and lime, and

Second: Removal of the soluble basic compounds by passing the clarified sewage through a bed of base exchange zeolite.

The time of retention of the sewage is, therefore, comparatively short, being in the Chicago plant only two hours, of which 1½ hours were spent in the clarifier. Two auxiliary operations—first, filtration and incineration of the sludge with recovery of the iron for further use, and second, regeneration of the zeolite and recovery of the basic nitrogen, principally as ammonia, complete the process.

The experimental plant, which was constructed entirely of steel, was located in the grit chamber building of the North Side Works, on a space 36 feet long by 14 feet wide.

Incoming sewage from the pumping station passed through the grit chambers at the rate of about 9 inches per second and thence through the coarse screens. By these two operations a large part of the grit and heavy solids was removed.

Sewage, after having passed the aforementioned pieces of apparatus, was pumped to a flow control box of the demonstration plant, from which it flowed by gravity through the following units:

An iron dosing tank 18 inches square by 5 feet long. A ferric sulphate solution, containing 50 grams per litre of iron, was added to the head of this tank. Air agitation was used to disperse the soluble iron promptly and thoroughly, as well as to oxidize any ferrous iron to the

ferric state and keep it in such condition. The time of retention was about 4 minutes and the rate of forward motion of the sewage was about 1 foot per minute. The pH of the sewage leaving the tank was about 6.5.

A lime dosing tank 18 inches square by 10 feet long, where milk of lime, containing 50 grams per litre of CaO, was added and thoroughly mixed by air agitation. Sufficient lime was added to bring the pH of the sewage to about 7.5; the time of retention was 8 minutes. The amount of air used in both the iron dosing tank and the lime dosing tank was .045 cubic feet per gallon.

A clarifier 6 feet in diameter by 8 feet deep. The sewage was introduced along one-third of the circumference wall at a point 6 inches above the bottom, passed upward through a blanket of sludge and overflowed on the opposite side after passing under a baffle board. The sludge was moved by a Dorr type rake to the central outlet. The capacity of the clarifier was about 1,000 gallons per square foot per day and the time of retention was 1½ hours.

Two zeolite reactors, each 36 inches long by 27 inches wide with 13 cubic feet of 10 to 30 mesh zeolite in a bed about 22 inches deep. Only one was used at a time for treatment of sewage, the other being out of service for backwashing and regeneration. The sewage flow was downward through the bed and the rate was .6 foot per minute. The capacity of the unit was 180,000,000 gallons per acre per day. The effluent from the zeolite was the purified sewage.

Sludge Disposal

The sludge was pumped periodically from the clarifier to a thickener, 3 feet in diameter by 3 feet deep, equipped with a Dorr raking mechanism. The overflow from the thickener ran by gravity to the iron dosing tank. The thickened sludge flowed by gravity to a 1 foot by 1 foot Oliver filter. The filtrate was returned to the system for treatment and the filter cake, which was brought to about 75 per cent moisture, was ready for complete disposal by incineration.

The incineration unit was a 2-section piece of apparatus consisting of a drier and an incinerator. The drier consisted of a rotating pipe, 1 foot in diameter by 6 feet long, insulated with slag wool and was heated by waste gases from the incinerator, the incoming temperature of which was 675° to 700° C., while the outgoing temperature was about 125° C. The moisture content of the sludge was reduced from 75 per cent to less than 8 per cent.

The incinerator was a steel shell, 19 inches in diameter and 4½ feet long, having a refractory lining giving an effective diameter of 10 inches. The heat was supplied by a blast burner using city gas. Charging was

done continuously by means of a screw conveyor and discharging was by hand. In both sections of the unit, the sludge and the gases traveled concurrently, with the result that, in the incinerator, the products of combustion had to pass over highly heated ash rich in iron oxide, while, in the drier, the hottest gases met the wettest cake. The heat value of the sludge was about 4,800 B.T.U. per pound dry weight.

The incinerator ash, containing from 30 per cent to 32 per cent iron oxide, was transferred to a 2-gallon cast iron kettle, where it was treated with 66° Bé sulphuric acid at 150° C. for one hour. This product was leached with water in a rubber-lined steel tank and the insoluble residue was removed in a stoneware filter. The ferric sulphate solution, diluted to proper strength, was sent to the iron feed tank and the innocuous residue, consisting largely of silica, was discarded.

Disposal of Soluble Putrescible Material

By a base exchange reaction, the soluble basic nitrogenous salts were concentrated in the zeolite bed. In addition, the small amount of suspended solids in the overflow was removed. At intervals of about 24 hours, the zeolite reactor was taken out of service and the filtered suspended solids were removed by pumping the purified sewage upward through the bed. The backwash flowed to a tank where the sludge was settled out and pumped to the iron dosing tank, while the supernatant liquid ran to waste. The bed was allowed to drain and a brine solution of 200 grams per litre was pumped from storage to the bed, where it percolated downward through the zeolite at the rate of 1.8 feet per minute. The volume of brine used was 300 gallons, or about 1 per cent of the flow, and the time of percolation was 2 hours. After the brine had finally passed through, some clarifier overflow was used as a replacement wash to recover the salt and the zeolite bed was allowed to stand under water until needed.

The spent brine was pumped to a head tank of about 350 gallons capacity, from which it ran by gravity to a mixing tank, where lime was added to liberate the NH_3 from its salts in the form of NH.OH. It then flowed to a settler, where the precipitated solids, principally magnesium hydroxide, settled out, and continued through a heat interchanger to the middle point of a packed rectifying tower, 1 foot in diameter by 5 feet high. The solution passed down the tower, where it met vapors rising from a gas-fired boiler, 1 foot wide by 2½ feet long. The NH_4OH was driven off and condensed, while the stripped solution, containing about 0.10 per cent to 0.15 per cent ammonia, passed through a heat exchanger to a storage tank for use again in regenerating zeolites. The ammonia was condensed in a water cooled S-bend condenser, while the

solids obtained from the settler were filtered off and discarded. The concentration of the NH_3 in the incoming salt solution was 1.5 grams per litre, while that of the recovered ammonia solution was about 30 grams per litre.

Operating Details

The operation of the plant was comparatively simple. In the six months that it ran, about 6,000,000 gallons of sewage were treated with no shut-downs.

One man took care of the entire sewage purification section. His duties consisted of watching certain points of the operation to be sure that there was no stoppage of the flow of sewage, checking the iron feed once an hour to prevent any variation and regulating the lime flow so that a constant pH was kept at the clarifier inlet. He also pumped sludge from the clarifier when the height of the sludge blanket made it necessary, and regulated the outlet valve of the zeolite filter to compensate for increased resistance in flow due to the deposit of a small amount of solids on the bed. In addition, he regulated the flow of thickened sludge into the Oliver filter. In the design of a large plant, obviously a number of the manually controlled features will be taken care of by automatic feed and control equipment.

The ammonia recovery, the incinerator and iron recovery units were operated only on the day shift, each having its individual operator. These two operators were necessary because the use of automatic devices in such small sized units was impracticable.

The flow of sewage through the plant was kept fairly constant throughout the period. However, for a certain period of time, the addition of coagulating materials was altered with the hourly or daily change in the strength of the sewage. The rates of feed for the iron varied from 20 to 45 p.p.m., but no improved results were obtained with quantities greater than 25 p.p.m. For the last two weeks of operation, the amount of iron added for coagulation and lime for pH control was drastically reduced and the results indicated that high degrees of purification could be obtained with far less amounts of coagulants than were used in the greater part of the experimental run. The amount of lime that was added was governed entirely by the pH of the sewage as it entered the clarifier. Different pHs between 7.0 and 7.6 were tried without any apparent effect on the clarifier operation. However, the higher pH was adopted to protect the zeolite.

The optimum height of the sludge blanket in the clarifier was found to be 3 feet from the top and the sludge level was maintained at that point by varying the rate of pumping.

The zeolite beds were alternated every 24 hours and gave a highly satisfactory effluent. Periodically the ammonia content of the effluent was shown qualitatively by the Nessler test. Occasionally it was found necessary to backwash the filter during the run to remove excess solid material carried over from the clarifier or to remove air introduced if the liquid level was accidentally allowed to fall below the top of the bed. A great majority of the time, the total volume of the backwash was less than 2½ per cent and rarely exceeded that figure. Because the sewage was hard, we were not certain of its effect upon the zeolite, but we were pleased to find that the types of zeolite used stood up very well in spite of the high lime and magnesia content of the sewage. Except for a negligible amount of zeolite lost during the initial period of operation, the same zeolite was used in the filters throughout the entire demonstration.

Probably the greatest individual improvement in the process was made in the filtration of the sludge. We were able to obtain a moisture content almost exactly as we predicted, but the rate of filtering in the last month was 9 times as high as in the first month and this high capacity was reached by progressive stages. The addition of about 40 pounds of lime per million gallons was largely responsible for the high filtration rate. At first lime was added to the thickener but this was ineffective, and subsequently it was added to the filter trough.

The incinerator and drier performed in a very satisfactory manner. The amount of gas required per unit was about $\frac{1}{10}$ of that actually used in New York, a fact which strengthens our belief that, except for ignition, large scale units will not require heat other than that supplied by combustion of the sludge. The drier discharged at a much lower moisture content than in the New York experiments; the dried particles were of uniform size, about ¼ inch mesh, and in such condition that they were fed to the incinerator by a 2-inch screw conveyor.

There is no particular comment to make on the operation of the ammonia plant except to say that it demonstrated satisfactorily the practicability of recovering ammonia and regenerating the brine. However, the basic nitrogen obtained was in the form of ammonium hydroxide, whereas in a large plant, which would be of different type than that used in the experimental plant, the ammonia would be recovered as liquid NH_3.

During the period of operation, conditions varied over a wide range. There were extensive rainy spells as well as periods of very hot weather and, in several instances in the early operation, the temperature was below freezing. On one particular occasion, there was a bad oil leak at a foundry and the oil had to be handled by the plant for a period of three

SUMMARY OF RESULTS OBTAINED BY CHEMICAL PROCESS AND BY ACTIVATED SLUDGE PROCESS, NORTH SIDE SEWAGE TREATMENT WORKS

	1933							
	March	April	May	June	July	Aug.	Sept.	Average
B. O. D.								
Raw Sewage	98.5	100.6	77.4	112.0	94.0	113.8	123.0	103.0
Chemical Effluent	11.5	11.7	8.0	10.4	8.3	8.2	6.1	9.1
Per Cent Removal	88.3	88.4	89.7	90.7	91.2	92.8	95.0	91.2
Activated Effluent	12.9	11.9	11.0	13.4	7.4	12.6	10.0	11.3
Per Cent Removal	86.9	88.2	86.2	88.0	92.1	88.9	91.9	89.0
SUSPENDED SOLIDS								
Raw Sewage	135.0	127.4	120.0	146.0	119.0	135.0	151.0	133.0
Chemical Effluent	1.8	1.5	1.5	3.6	4.8	5.3	5.2	3.9
Per Cent Removal	98.7	99.6	98.8	97.5	96.0	96.1	96.6	97.4
Activated Effluent	14.6	15.8	11.0	14.1	9.3	14.2	9.7	12.7
Per Cent Removal	89.2	87.6	90.8	90.3	92.2	89.5	93.6	90.4
ORGANIC NITROGEN								
Raw Sewage	9.5	8.7	7.7	9.3	7.7	8.7	9.1	8.7
Chemical Effluent	1.9	2.1	2.0	1.6	1.8	1.5	1.5	1.8
Per Cent Removal	80.0	75.6	74.0	82.8	76.6	82.8	83.5	79.3
Activated Effluent	2.0	2.3	2.7	2.2	2.3	2.0	2.0	2.2
Per Cent Removal	78.9	73.6	64.9	76.3	70.1	77.0	78.0	74.7
AMMONIA NITROGEN								
Raw Sewage	6.9	6.6	5.5	7.9	7.7	8.6	8.5	7.5
Chemical Effluent	1.9	2.3	1.5	3.1	3.2	2.8	2.4	2.5
Per Cent Removal	72.5	66.0	73.0	61.0	58.4	67.5	72.0	67.4
Activated Effluent	3.9	3.8	3.1	4.2	3.9	3.0	2.9	3.5
Per Cent Removal	43.5	42.4	43.6	46.8	50.0	65.1	65.9	51.0
CHEMICALS								
Iron, p.p.m.	32.1	31.3	38.5	27.0	24.5	26.2	19.5	28.4
Lime, p.p.m.	43.1	45.8	52.4	49.4	46.7	52.9	31.6	44.5
Salt, lb. per m.g.				840.	800.			1000.
H_2SO_4, lb. per m.g.			1180.	875.	865.	918.	1030.	970.
Lime, lb. per m.g.			640.	670.	621.	637.	484.	600.
SLUDGE DISPOSAL AND FILTRATION								
Per Cent Solids:								
Clar. Sludge	5.2	3.9	4.0	4.3	6.5	4.8	3.8	4.8
Thickener	6.1	5.7	6.7	7.5	8.8	9.1	7.3	7.3
Per Cent H_2O Filter Cake	79.1	75.6	74.4	76.7	75.8	77.0	77.0	76.4
Dry Solids/sq. ft./hour, lb.	.7	1.4	3.0	3.2	4.7	6.1	6.0	3.6
Solids/m.g., lb.	1837.	1528.	2080.	1450.	1460.	2080.	1950.	1770.

BACTERIA REMOVAL—Average of 13 counts taken during April, May, June and July:

	Colonies per c.c.	
	Total Bacteria 37°	B. Coli
Raw Sewage	850,000	100,000
Chemical Effluent	22,000	2,300
Per Cent Removal	97.5	97.7

days. The only inconvenience we suffered was that we had to wash the zeolite filters slightly more often than when the normal flow of domestic sewage was being treated in the plant. On some occasions it was found necessary to pump activated sludge back to the head of the treatment works and the demonstration plant was obliged to treat sewage very high in suspended solids. In every case, as the results indicate, the incoming sewage was handled and purified in a highly satisfactory manner. These instances, together with others too numerous to mention, confirm our belief that the process is not sensitive to either conditions of weather or variations in the type of sewage.

This plant was operated by workmen from the North Side Sewage Treatment Works, all samples were taken by Sanitary District employees under the direction of Dr. F. W. Mohlman, and all analyses were made by the chemical department of the Sanitary District.

Estimated Costs of Construction and Operation

On the following pages are two estimates of costs of operation, one of which has been prepared for The Sanitary District of Chicago, and the other at the request of Mr. Ernest P. Goodrich, Commissioner of Sanitation of the City of New York.

Construction costs covering a wide range of sizes have also been prepared at the request of the Commissioner of Sanitation of the City of New York.

All cost figures are exclusive of royalty.

Chemical Purification Process

capacities: 3—6—12—24 m.g.d.

Tabulation of Estimates
Total Construction Costs

	3 M.G.D.		6 M.G.D.		12 M.G.D.		24 M.G.D.	
	Cost $	%	Cost $	%	Cost $	%	Cost $	%
Excavation	5,000	2.4	8,000	2.7	16,000	3.2	30,000	3.5
Concrete	16,000	7.6	28,800	9.8	48,000	9.8	80,000	9.3
Superstructure	25,000	11.8	30,000	10.2	48,000	9.8	72,000	8.4
Mechanical and Electrical Equipment	97,300	46.1	134,000	45.4	223,300	45.4	390,400	46.8
Pipes, Valves and Fittings	9,000	4.2	14,000	4.7	25,000	5.1	60,000	7.0
Power Circuits	5,000	2.4	6,500	2.2	7,500	1.5	9,000	1.1
Illumination	1,600	0.7	1,800	0.6	3,500	0.7	5,000	0.6
Miscellaneous	8,000	3.8	10,100	3.4	17,000	3.5	20,000	2.3
Total	166,900	79.0	233,200	79.0	388,300	79.0	666,400	79.0
General Items: 26½% of above	44,100	21.0	61,800	21.0	102,700	21.0	176,600	21.0
Construction Cost Grand Total	211,000	100.0	295,000	100.0	491,000	100.0	843,000	100.0
Construction Cost per M.G.	70,300		49,200		40,900		35,125	

Chemical Purification Process

50 million gallons daily

Plant Construction Cost Estimate

Material and Labor Cost Summary:

Bar Screens	$38,000
Grit Chambers	25,500
Fine Screens	72,000
Main Sewage Pumps	52,000
Iron and Lime Mixing Tanks	135,000
Clarifiers	170,200
Zeolite Reactors	277,800
Ammonia Recovery	185,000
Sludge Filters	75,000
Incinerators	134,600
Acid Leaching	91,100
Boiler Plant	51,100
Miscellaneous Construction	141,500
Total—Exclusive of General Items	$1,448,800

General Items:			
Engineering	5.0%		
Job Supervision	2.0		
Insurance	2.0		
Temporary Construction	1.0		
Construction Equipment	1.5	11.5%	
Contingencies	5.0%	6.0	
Compensation	8.0	9.0	
		26.5%	
Total—General Items			383,900
Construction Cost—Grand Total			$1,832,700
Construction Cost—Per M.G.D.			$ 36,654

Chemical Purification Process

100 million gallons daily

Plant Construction Cost Estimate

Material and Labor Cost Summary:		
Bar Screens	$50,000	
Grit Chambers	46,000	
Fine Screens	130,000	
Main Sewage Pumps	76,000	
Iron and Lime Mixing Tanks	240,000	
Clarifiers	260,000	
Zeolite Reactors	525,000	
Ammonia Recovery	315,000	
Sludge Filters	115,000	
Incinerators	218,000	
Acid Leaching	125,000	
Boiler Plant	83,000	
Miscellaneous Construction	208,000	
Total—Exclusive of General Items		$2,391,000

General Items:			
Engineering	5.0%		
Job Supervision	2.0		
Insurance	2.0		
Temporary Construction	1.0		
Construction Equipment	1.5	11.5%	
Contingencies	5.0%	6.0	
Compensation	8.0	9.0	
		26.5%	
Total—General Items			634,000
Construction Cost—Grand Total			$3,025,000
Construction Cost—Per M.G.D.			$ 30,250

New York Estimates

OPERATING COST PER M. G.

SUMMARY

	3 M.G.D.	6 M.G.D.	12 M.G.D.	24 M.G.D.	50 M.G.D.	100 M.G.D.
Operators and Supervision	$12.13	$ 8.13	$ 6.73	$ 4.41	$ 4.14	$ 2.87
Maintenance: Labor and Material	4.60	3.31	3.29	2.69	2.33	1.82
Chemicals	9.24	9.24	9.24	9.24	9.24	9.24
Power and Illumination	5.30	4.18	3.46	2.82	2.71	2.46
Steam	5.31	4.52	4.18	3.78	—	—
Gas	0.26	0.26	0.26	0.26	—	—
Fuel	—	—	—	—	2.00	1.92
Operating Cost—Total	36.84	29.64	27.16	23.20	20.42	18.31
Capital Cost (4% interest and 4% amortization)	15.40	10.80	9.00	7.83	8.00	6.63
	$52.24	$40.44	$36.16	$31.03	$28.42	$24.94

This estimate does not include possible iron and zeolite losses, ammonia credit, nor credit for recuperation of excess heat from incineration.

Chicago Sanitary District

AVERAGE DAILY CAPACITY—136 MILLION GALLONS

OPERATING COST PER M.G.

Summary:

Supervision and Operation	$ 5.50
Maintenance: Labor and Material	2.32
Chemicals	9.12
Power and Illumination	1.46
Fuel	1.40
Miscellaneous Material for Office and Laboratory	.15
Operating Cost—Total	$19.95

Not Included: Credit for ammonia or recuperation of excess heat from incineration.

Supervision:

	Total Monthly Salary	Total Monthly Salaries
1 Superintendent	$400	$ 400
1 Assistant Superintendent	300	300
2 Junior Assistants	225	450
1 Clerk	250	250
1 Chief Chemist	350	350
1 Analytical Chemist	300	300
1 Junior Chemist	200	200
1 Laboratory Helper	135	135
Supervision—Total		$2,385
Supervision—Per M.G.		$ .60

Operation	Monthly Salaries					
	$395 Chief Op. Engr.	$350 Operat. Engr.	$225 Foreman	$180 Helpers	Total No. of Men	Total Monthly Sal.
General Plant	1	—	4	8	13	$2,735
Screening	—	—	4	4	8	1,620
Main Sewage Pumping	—	4	—	4	8	1,920
Iron Mixing and Clarification	—	—	4	4	8	1,620
Zeolite	—	4	4	8	16	3,540
Ammonia Recovery	—	—	4	4	8	1,620
Filtration	—	—	—	4	4	720
Incineration	—	4	4	4	12	2,820
Acid Leaching	—	—	—	4	4	720
Power Distribution	—	1	4	—	5	1,200
Boiler Plant	—	—	4	4	8	1,620

Number of Men—Total	94
Operation—Total	$20,135
Operation—Per M.G.	$ 4.93

Maintenance:		
Labor:	Total Monthly Salary	Total Monthly Salaries
1 Shop Foreman	$350	$ 350
3 Electricians	300	900
3 Mechanics	265	795
3 Pipe Fitters	265	795
6 Helpers	150	900
Maintenance—Labor Total		$3,740

Material:

The cost of maintenance materials is estimated to be 1½% of total construction cost yearly or 1½% of $35,000.

Maintenance—Material Total	$525
Labor per M.G.	$ .92
Material per M.G.	1.40
Maintenance—Per M.G.	$ 2.32

	Quantity per M.G. Lbs.	Cost per Ton	Cost per M.G.
Chemicals:			
66° Bé Sulphuric Acid	960	$9.00	$4.32
Ground Lime	680	7.00	2.38
Salt	840	5.75	2.42
Chemicals—Per M.G.			$9.12

Power and Illumination:

Installed H.P. Total	1,970	
Average H.P. Load	833	
Expected Efficiency Average	82	
K.W.H. per M.G.—Average	135	

Illumination:

Estimated per M.G.—Average	12 K.W.H.	
Electric Rate	.85¢ per K.W.H.	
Power per M.G.	160 K.W.H.	$1.36
Illumination per M.G.	12 K.W.H.	.10
Power and Illumination—per M.G.		$1.46

Fuel:

Ammonia Recovery	
Steam required per lb. of Ammonia is 85 lbs.—with 100 lbs. of Ammonia per M.G.	8,500 lbs.
Steam required for Building purposes	
With a total of approximately three million cu. ft. of building space and taking 3 B.T.U. per cu. ft. per hour as the heat required during 6 months, the result per M.G. average total is	1,000 lbs.
Total Steam Required	9,500 lbs.
say	10,000 lbs.
Cost of Coal	$2.80 per ton
Cost of Fuel—per M.G.	$1.40

In conclusion, we wish to express our appreciation for the hearty co-operation and assistance given us by The Sanitary District of Chicago, especially by

MR. PHILIP HARRINGTON — DR. F. W. MOHLMAN
MR. LANGDON PEARSE — MR. WALTER M. BELL

as well as by every member of the staff with whom we came in contact.

DISCUSSION

Langdon Pearse; *Sanitary Engineer, The Sanitary District of Chicago:* I regret exceedingly that I am unable to present a discussion of this interesting paper in person. The copy of the paper arrived so late that I shall not attempt at this time to cover all the points, reserving that for a more complete presentation, if opportunity permits.

Messrs. Gleason and Loonam have described clearly the tests conducted by the Sanitary District of Chicago engineering staff on the Guggenheim process at the North Side Works.

In the tabulation of results on page 456, the 5-day B.O.D. of the chemical effluent appears to be intended for the unseeded tests. The monthly averages given therein for March and September do not agree with the log sheets, but should be 12.5 and 7.5 p.p.m. respectively, bringing the

average up to 9.5 p.p.m. The seeded results should be more indicative of the true B.O.D. On this basis the 5-day B.O.D. of the chemical effluent from April to September inclusive averages 13.0 p.p.m., including, however, the rather high figure of 26.6 p.p.m. in April.

In the comparison of results with the activated sludge effluent, it should be noted that the Guggenheim process was being demonstrated to bring out its highest efficiency, whereas the North Side Works using the activated sludge process was being operated with strict economy, throughout the period of these experiments without attempting a high degree of efficiency. As a result, the tabulation shows removals of B.O.D. in the activated sludge which are somewhat lower than those which can be attained. I believe it entirely fair to say that if the activated sludge plant had been operated to demonstrate the best it could do the removal of B.O.D. would in general have exceeded that of the Guggenheim process. As to removal of suspended matter, the Guggenheim process has shown some advantage. On the removal of ammonia from June to September, there is little difference.

In the cost data given, it would be desirable for Messrs. Gleason and Loonam to state the unit prices of chemicals, as well as the amounts per million gallons, used in making up the figures. Over a term of years, a considerable variation in chemical costs appears likely to occur. The record of power costs on activated sludge would indicate a more stable situation.

The authors fail to note that from time to time nitrate of soda was used, varying from 45 to 141 lb. per million gallons. This was claimed to aid in keeping the sludge in better condition.

The cost figures given are those prepared by the Guggenheim staff. It would, therefore, be of interest if the authors would state for what locality the figures given are intended.

No allowance is shown in the estimate for royalty.

My personal opinion of the present status of the Guggenheim process may be summed up as follows:

1. Of the various so-called chemical processes it appears to be the most complete, offering an entire process, capable of delivering a high-grade effluent and disposing of the sludge.

2. To determine its sufficiency and efficiency under actual conditions, an actual large scale plant is desirable, to settle questions of control, adaptability, economy of chemicals, recovery of by-products, and freedom from odor from the sludge incineration.

3. In comparison with other processes, first cost and annual cost are important. The relation may vary in different localities, and in the size of plant. General comparisons are difficult to make. Consequently, I feel that the discussion of cost should be approached with caution.

4. The future of the process would seem to hinge in part on other factors:

(*a*) The cost and feasibility of the regeneration of ferric sulphate.
(*b*) The recovery of ammonia and the income to be derived therefrom.
(*c*) The royalty charged for the use of the process.

It would seem as though there may be situations where the process may be adaptable, especially on more concentrated sewage and in soft water regions.

In concluding, I wish to congratulate the Guggenheim Brothers and their staff and in particular Major Gleason on the scientific demonstration made at the North Side Works of the possibilities of their process.

Alexander Potter; *Consulting Engineer, 50 Church St., New York City:* This paper represents an outstanding contribution to scientific knowledge in this renaissance period of chemical sewage treatment.

This process is of particular interest in that it covers the entire range of sewage treatment and sludge disposal in a seemingly practical and reasonably economical manner. The process is unusually complete, providing, as it does, for as thorough treatment of the sewage as has ever been attained by any biological method, except by intermittent sand filtration, and at the same time providing, in the experiment, for carrying sludge handling further than has heretofore commercially been attained.

Our operating data of the test plants with chemical treatment in Atlanta, Georgia, for a period of six months, and in Philadelphia, Pa., during October and November, justify comment upon several of the salient points in the authors' paper.

One of the first points of outstanding importance is the question of efficiency of treatment. The data from Chicago indicate removals of 91.2 per cent of the 5-day B.O.D., 97.4 per cent of the suspended solids, and 79.3 per cent organic nitrogen. We have found that an efficiency of removal reasonably approaching these values can be obtained even without filtration, and that with rapid sand filtration efficiencies approaching very closely those obtained in Chicago are possible—all within an economical range of chemical feed. The writer is accordingly certain that the Guggenheim process with the zeolite filter as an adjunct, can undoubtedly perform the work demonstrated at Chicago. It is also probable that with a heavier raw sewage than prevailed at the Chicago plant, efficiencies even higher than those above mentioned could be obtained.

Inasmuch as these results are as effective as can normally be obtained by the activated sludge method of treatment, the authors' work represents an extremely important contribution in the field of sewage treatment.

A second point of importance brought out by the authors is in relation to the facility with which the chemical plant operated satisfactorily with radical variations encountered in sewage character. This conclusion is effectively supported by results of operation at Atlanta and at Philadelphia. In Atlanta, particularly, the sewage treated was from combined systems, and radical variations in strength occurred not only from day to day but from hour to hour, not only from change in concentration because of storm water with its infiltration lag, but due more particularly to the presence of industrial wastes which at times caused variation in the pH of the raw sewage ranging from 2.2 to 11.0. The ability of chemical treatment to handle such wide variations successfully represents a strong point in its favor, as similar variations would upset the proper functioning of a biological plant for substantial periods of time.

A third point mentioned by the authors is the greater ease with which chemical sludge can be handled on a vacuum filter, together with the comparatively high rate of loading on the filter surface. The writer believes it may be pertinent to point out that the ease of filtering chemical sludge also reflects the ease of dewatering chemical sludge by other methods, including the old but more commonly adopted method of handling on drying beds. The increase in filter loading from .7 lb. net dry solids per sq. ft. of filter per hour during March, to 6.0 lb. in September, is most interesting. Similar improvement in loading was obtained by acquiring familiarity with proper conditioning of the sludge in our Atlanta tests.

Another point of interest in the paper is in relation to the data on weight of dry solids obtained and expressed in pounds net dry solids yield per million gallons. The average for the six months operating period is stated to be 1,770 lb. per M.G. In a series of tests in Atlanta involving the treatment of a total volume of approximately 20 M.G. of sewage for this particular test, the average dry solids was 1,965 lb. net dry solids per million gallons. These relative results are substantially in the ratio of suspended solids in the raw sewage, namely 133 p.p.m. in Chicago and 159 p.p.m. in Atlanta.

These values of less than 2,000 lb. per M.G. are illuminating when compared with the weight normally reported to be obtained from chemical precipitation, namely 3,000 to 4,000 lb. per million gallons.

To recapitulate, the Chicago experiment demonstrates a method of treating sewage in which efficiencies equal to the best that can be obtained by activated sludge have actually been obtained, and that chemical treatment produces a sludge which not only involves a volume of approximately one-sixth of the volume produced by activated sludge, but of still greater importance, with chemical treatment the sludge is of

an entirely different character and more favorable for economical disposal.

I would be very much interested to have the authors' results obtained by their process up to the point of filtration through the zeolite, as in my opinion there are far more instances where a medium degree of treatment is required than where the extremely high degree of treatment obtained by the complete process would be justified, at least throughout the entire twelve months period.

We have not had an opportunity to consider how conservative are the estimates of cost of construction and operation of the Guggenheim process. It would indeed be marvelous if in actual construction, variations in different degrees in cost of some or all of the important component parts of the authors' smooth curves of various sized plants, will not be found. This fact, coupled with the estimates made by the writer for special occasions, which vary substantially from the authors' figures for similar construction, prevents acceptance of the authors' estimates for general application.

As to the wisdom of a more general adoption of sewage treatment plants capable of producing such highly satisfactory results as shown in the authors' paper, I am not prepared to state, but considering the activities of Izaak Walton clubs and promulgations of edicts like that of the late Dr. Dixon to the effect that it was his intention to restore the streams of Pennsylvania to their pristine purity, the time may come when the highest degree of purification possible will be demanded everywhere.

If these and similar forces should demand the highest degree of purification, it must be conceded that the same degree of purification will not be required during the entire year because of the varied absorptive properties of receiving waters. The authors have shown, and other investigators have proved, that chemical treatment lends itself most readily to meet these fluctuating conditions. Furthermore, the constantly changing character of the sewage from time to time and hour by hour properly demands flexibility in methods of treatment, which is most readily accomplished in a chemical plant.

In the consideration of any specific type of treatment, the final action for any particular city must be based on a comparison of simplicity of design, of cost of construction and operation, flexibility, efficiency, demonstrated purification results and relative license fee where patents may be involved.

Wellington Donaldson; *With Fuller and McClintock, 11 Park Place, New York City:* Major Gleason's paper furnishes an adequate answer to questions in the minds of sanitary engineers, chemists and plant operators, since last year's meeting, as to the actual performance of the pilot

plant of the Guggenheim Process which has been in operation at the North Side Works in Chicago. The results speak for themselves. This demonstration plant of some 25,000 gallons per day capacity has been operating in parallel with one of the country's best activated sludge plants on the same sewage for six months without a shut-down. Analyses made by an impartial and competent laboratory indicate an appreciable and consistent superiority of the chemical effluent over the activated sludge as to appearance, also a slight superiority as to B.O.D. and organic nitrogen content.

The performance being satisfactory and the operation reliable, the next important element for careful consideration is the probable cost in investment and operation of a sizable municipal installation. The cost figures presented by Major Gleason have doubtless been worked up with meticulous care and conservatism for hypothetical plants ranging in capacity from 3 to 136 m.g.d., but not until practical scale plants are built and in operation for a time in the hands of the average municipal personnel can costs be actually determined—and frequently not then. However, Major Gleason's estimates must stand as a challenge when consideration is given to completeness of treatment, freedom from nuisance and prompt disposal of all sewage material in a compact structure.

In comparing the operations of the two processes at Chicago it should be borne in mind that the chemical effluent was filtered and the activated effluent not filtered. It is generally recognized that the major portion of the B.O.D. in an activated sludge effluent is resident not in the liquid phase, but in the light flocculent suspended particles which cannot be completely held in the final clarifier. Filtration of activated sludge effluent, of course, is not without precedent. Rapid sand filters have been used to "polish" the activated sludge effluent at Grand Canyon, Ariz., since 1926. At Tenafly, N. J., the activated effluent has been passed through slow sand filters since the early part of 1928, while at Barrington, N. J., rapid sand filters have been in operation since 1931. The Rockville Centre, L. I., plant has been filtering its settled activated sludge effluent for about two months through vacuum filters, using paper pulp as a filter medium.

These remarks are pertinent only as indicating that the activated sludge process is capable of being pushed to a much higher degree of purification than is usually associated with it, because the majority of installations do not include filtration of the effluent.

Major Gleason and his associates are to be congratulated upon the manner in which their enterprise has been projected. It is somewhat unusual in the sewage treatment field to witness new processes being subjected first to painstaking research study, then to thorough tests by disinterested parties, prior to commercial exploitation. Apparently the

policy followed here has been to develop carefully the facts and lay them upon the table.

George H. Gleason; *Closure Memorandum:* Regarding Mr. Langdon Pearse's comments on the discrepancies between this paper and the blueprinted sheets, we are not very far apart. The difference is due to the fact that in this paper I took the official starting date of the run as March 6th and the official closing date as September 17th. Assuming the date of March 6th, the B.O.D. is 11.5 instead of 12.5, as pointed out by Mr. Pearse. The same applies to the month of September. Based on September 17th as the closing date, the B.O.D. is 6.1.

Regarding the seeded B.O.D.'s, they are not very far apart from the unseeded, except in one case, the month of April. Eliminating this one month, which Dr. Mohlman thought might be in error, the seeded B.O.D. ran 10.3, showing a difference of only about 1 p.p.m. between the seeded and unseeded.

Regarding the costs of chemicals, this is specified both as to consumption per million gallons, unit cost and cost per million gallons in the paper.

Sewage Works Operation

THE MECHANIZATION OF SEWAGE TREATMENT*

By Edmund B. Besselievre

Sanitary Engineer, The Dorr Co., Inc., New York

It is the purpose of this paper to record the developments in the mechanization of sewage treatment in their historical sequence as the various units were perfected, and to describe generally the salient features and benefits of mechanical appliances in each phase.

Authorities are in general agreement that the first serious thought was given to the treatment of sewage to prevent the continued pollution of streams at the time of the formation of the first Commission on Town Sewage Disposal in England in 1857 and in its first report in 1858 recommending the treatment of sewage by irrigation on land. For almost the entire balance of the 19th century the concentration of effort was in the development of satisfactory processes and not much attention was given to the mechanics of handling the by-products of these processes, except in a few isolated cases. It was during this time that the contact filter, the trickling filter and chemical precipitation were evolved, and experimental work was done at Lawrence, Mass., and in England on the use of air as an oxidizing agent.

These earlier plants and experimental units were practically all based upon the use of natural principles of hydraulics and common structures with hand operation to control the flow of sewage and the handling of its by-products. Little or no thought seems to have been given to attempting any measure of mechanical control until almost the beginning of the twentieth century.

As the knowledge of the characteristics and behavior of sewage under given conditions became widespread, it became apparent that many of the operations of removing solids from the sewage by hand were expensive and objectionable.

The modern conception of sewage treatment is that it is primarily a problem in "classification"—of segregating the various kinds of solids in groups, the basis of which is the greatest economy and facility of final disposal for each type of material.

* Presented before Joint Session, Public Health Engineering Section A. P. H. A. and Central States Sewage Works Assoc., Thursday, October 12, 1933, Indianapolis, Ind.

The earlier plants did not go into such great refinement in this regard as we do today. It was common practice to allow the sewage to flow into tanks with a simple bar rack at the entrance as a protection against the largest objects. The sedimentation units received the balance of the solids and were faced with the concurrent difficulties due to clogged sludge lines, heavy scum, etc.

It was the difficulties with tanks that concentrated the attention of engineers on means to eliminate future troubles of the same nature and led to the gradual development of one mechanical unit after another.

Pursuing the "classification theory" further to bring out the importance of this idea in simplifying and economizing sewage treatment we find that there are three general classes of solids in sewage, the amount of each varying in concentration dependent upon local conditions or the inclusion or exclusion of industrial wastes. These are:

1. The garbage type of solid. This is represented by fruit skins, bits of rag, paper, bone, tin cans, hair, dead animals and similar material which would not be readily acted upon by the biological or other processes in the plant and are not suitable for fill. This material may best be disposed of by burying or incineration.
2. The inert solid. This includes the grit and sand which enters the system through manholes, faulty sewers or roof drains, coal, etc. This material is also not acted upon biologically and simply acts as a space filler in sedimentation and digestion tanks. Removed from the system it may be used for fill in low ground around the plant.
3. The decomposable solids. This includes the fecal matter which is the basic troublesome constituent of sewage, and other organic matters which come into the system from kitchens, vegetable matters and finely divided fibrous materials. Removal of this matter is, of course, the main objective of sewage treatment and must be subjected to treatment to reduce it to an inert, inoffensive, odorless mass.

Each of these classes of solids has definite characteristics which make it possible to effect a separation. The first, usually solids of appreciable size, may be removed from the system by a barrier, which we know as a "bar screen"; the second, more finely divided, have a specific gravity greater than 1.0 and will settle readily to the bottom of a channel or tank; the third, of a lower specific gravity, require an appreciable period of time, averaging from 1 to 2 hours or more, to settle to the bottom of a tank.

Coarse, or Bar Screening.—The first mechanized operation in sewage treatment was coarse screening. Originally all plants were preceded by

some type of bar rack, usually consisting of a set of fixed bars, set across the channel, with spaces of two inches or more between the bars. These bars were intended to intercept all of the material of greater size than the spaces, and at intervals these collected solids were raked by hand to the top of the bars and deposited on the floor. Allowed to collect there they became obnoxious, collected flies and became potential sources of transmission of pollution to nearby residences. Disposal of screenings in most plants was undoubtedly the most unpleasant and most unsightly part of the plant operation. As raking was intermittent it was usual to provide a sufficient width of bar screen channel to permit a certain amount of clogging without unduly raising the elevation of the sewage. This increased the expense of plant construction and complicated the removal of screenings due to the large area of screen to be raked. In New York State, for instance, as in others, the State Board of Health makes the rule that bar screens to be cleaned by hand must provide an area below the flow line equal to 300 per cent of the area of the inlet sewer for combined sewers.

The removal of solids by a mechanically cleaned bar screen prevents, to a large degree, the formation of mats. In a hand raked screen these solids are carried up with the rake and complicate the disposal problem. In a mechanical unit the openings between the bars are kept freer and thus allow the passage of the bulk of the organic solids, especially those of a fecal nature, for the treatment of which the sedimentation and other units are provided. To illustrate this difference between hand and mechanically operated screens we may cite Trenton, New Jersey. At the time of construction of this plant, about 1926, a hand raked bar screen with 2 inch spaces was installed. In 1929, a mechanical rake was installed, the bars and openings remaining the same. Records kept by the operator for both types of operation show that with the hand raked bars the average volume of screenings per million gallons of sewage was 1.0 cu. ft., whereas after the installation of the mechanical rake it rose to an average of 2.0 cu. ft. The character of the screenings changed materially. Before mechanical raking there was a large percentage of fecal matter, but after the use of the mechanical rake the volume of screenings doubled but the content of fecal matter was very materially reduced.

At the Trenton plant the release of the man who formerly raked the screen made a saving of 7 man hours per day, costing $3.30. The total saving in two years was sufficient to amortize the entire cost of installation of the mechanical raking unit.

At Worcester, Mass., where mechanical bar screen units were installed in November, 1931, displacing hand raked units, the tabulation of very carefully kept costs shows that in 1931 the cost of screenings

removal was $.301 per million gallons. In 1932, after an entire year of operation of the mechanical rake, the cost had dropped to $.162 per million gallons, a reduction of almost 50 per cent in one year.

The first mechanical rake recorded was installed at an underground station in Paris about 1867. France was the pioneer in this first step. The screen was a simple set of bars set in the channel with a chain propelled rake to drag the screenings to an upper platform. After this attempt others followed and we have a record of the installation at Wiesbaden, Germany, of a wing rack screen about 1899. One of the earliest English types was a band screen consisting of perforated plates, moved by a water wheel operated by the flow of sewage. These band screens are in use at several English plants today. A brush at the top of the band is used to remove the screenings that do not fall off of their own accord as the band turns over at the top of its travel.

In the United States this phase of sewage treatment was not given much serious consideration until about 1926, when the first mechanically cleaned unit was developed at Los Angeles by Van Norman. This unit consisted of a curved set of bars set in the channel and a revolving rake mechanism, which consisted of two arms, on the end of each being a set of rake teeth fitting between the bars of the screen. As this rake arm was turned by a motor, the screenings were carried up the bars and at the top were pushed off the raking plate by a spring operated push plate. These units have been installed in the U. S. at Los Angeles, Chicago, Winston-Salem and Durham, N. C., Fort Worth, Texas, and Aurora, Ill. A unit of a somewhat similar type has been installed at the Northeast Works at Philadelphia.

At the present time there are a number of different types in use in this country. The type most generally installed employs a raking mechanism which is an adaptation of the principle of the pantograph, whereby a driven gear transmits its motion in large scale to the end of the raking arm. Other types are straight bar units with a chain or wire rope propelled rake, which carries the material upward and discharges it at the top of its stroke.

Control of the first units was mainly by means of hand operated switches requiring the operator's attention for starting and stopping the rake, or it was allowed to run continuously. As there are many periods, especially during the night, when the volume of screenings is small, it soon became apparent that this was wasteful practice, both in power consumption and wear and tear. To meet this condition and provide a control a float-controlled switch element has been perfected, which operates on the principle of the differential head developed on a screen between the upstream and downstream sides. When sufficient screenings have accumulated on the bars to cause the level of the sewage in the

channel approaching the screen to rise, the float which is connected ahead of the screen rises and gradually tips the mercury switch and starts the screen rake. The screen rake then operates until the accumulation has been removed and the level ahead of the screen drops back to normal when the mercury switch is leveled, the contact is broken and the rake stops. A time element and brake is included in this control which stops the rake mechanism at the top of the stroke so that there is no chance for rags, etc., to accumulate on the rake. The floats ahead of and behind the screen are connected on free running rods or chains which permit the floats to ride on the surface of the sewage, so that regardless of the depth of flow in the channel the rise in head due to matting will operate the control. The loss of head may be regulated from two inches to six or more so that the period of screen operation may be controlled at will.

The essential feature in a mechanically cleaned screen is to provide adequate cleaning with least use of power. The bars should be spaced evenly and the rake provided with teeth which project entirely through the bars to prevent accumulations of screenings.

The bare cost of installing mechanically cleaned bar screens is usually greater than for plain hand raked bars, but there are a number of factors which usually show that the extra cost is more than offset by the benefits to be gained. Advantages of the mechanically cleaned type of screen are

1. The area of screen required for mechanical cleaning is usually not more than half that for hand cleaned units. In those cases where the screens are housed the saving in size of structure is of major importance.
2. Automatic float control saves labor.
3. Screenings may be raked up sufficiently high to be discharged directly into cans, carts or incinerators, eliminating handling, which is usually a nuisance and odorous with hand raked screens.
4. Power is saved where sewage is pumped, due to practical elimination of loss of head behind the screen.
5. Screenings contain less finely divided organic solids, making the disposal problem simpler and less messy.

In referring to coarse screens it is not considered within the scope of this paper to include the lifting basket type of screens used in the pumping stations at Boston, Washington, Philadelphia, etc. The reference is to screens with bars fixed in place in the sewage channel with mechanical means for removing the screenings from the bars.

A band type of coarse screen, known as the Brunotte screen, and developed in Germany, has been installed at St. Petersburg, Florida. This is a travelling belt made up of sections of vertical bars, with a

ledge at the bottom of each section to prevent screenings falling off during the upward passage. At the top of the travel the sections turn over and drop the screenings. Rubber fingers are also provided to remove any solids which tend to adhere to the bars.

Trickling Filters.—Adherence to historical sequence calls attention to the final oxidation step as the next recorded phase to receive attention. Prior to 1900 several forms of trickling filters using various materials for the filter medium had been employed, with splash plates for distributing the sewage to the bed, as well as various forms of spray nozzles, troughs, etc. English investigators were not satisfied with these and in 1894 Corbett experimented with a unit with revolving arms. Nothing further was done with this until about 1898 when the first full scale installation was made at Reigate, England, with a unit designed by Caink and manufactured by Candy. This unit consisted of a central column, connected to the inlet pipe, and provided with two or more arms provided with holes through which the sewage jetted, imparting a rotary motion to the arms. All of the rotary distributors developed since have employed the same principle, which provides for uniform distribution of the sewage over the bed surface. One variation was a unit driven by means of a 5 H.P. oil engine which was built and installed at Birmingham, England, by Scott-Moncrieff about 1900. Owing to the absence of operating cost of the jet propelled type, however, the motor driven type has never taken hold, except in cases of rectangular beds where the jet principle was not applicable.

These revolving distributors gained rapidly in favor in England, but were not installed in America until much later, due primarily to the fear that they would freeze in winter. The first recorded installation in this country was at Pontiac, Michigan, about 1920 by Hubbell. Potter about 1912 installed a travelling unit for rectangular beds at Springfield, Missouri. Several installations were made in Canada, notably at Toronto and Whitby, and were covered over to protect them against the extreme cold. Within the last three years there has been a revival of interest in these units and a number of installations have been made for both municipal, institutional and industrial plants.

The general advantages of the revolving distributor have always been present but the factors of economy of this unit seem to have been entirely neglected or overlooked for nearly two decades.

The revolving type of distributor essentially requires the construction of a circular bed, but in small plants this is no objection and in the larger plants the beds may be nested so that the minimum area is unused between them.

The salient features of this type of distribution are:

1. Avoids pumping of sewage where the head is insufficient for the fixed nozzle type of bed, which requires about 12 feet total head. The revolving type operates on a total head of less than 2 feet from high water in the dosing tank to the top of the stone in the filter bed.
2. Effect of wind is minimized. This also nullifies the disturbing effect of the wind.
3. One inlet pipe only is needed to bring the sewage to the central distributing column from which the arms take it.
4. The unit is more flexible to variations in flow. As the head in the dosing tank increases due to increased flows the unit will run slightly faster to accommodate itself to the extra impetus given by the jets, or will run longer to carry off the accumulation, but does not overdose any given area as in the case of the fixed nozzle type, which at its maximum spread of spray applies sewage mainly to the perimeter of the spray zone for a longer period and overdoses that area.
5. Less chance of dissemination of odors through the surrounding atmosphere as the spray is directed downward toward the stone and not sprayed up into the air where air currents can catch it.

Fine Screens.—A revolving wing screen was installed by Schneppendahl at Wiesbaden about 1899. This was followed by experimentation with a revolving disc by Reinsch, which was installed at Halle, Germany, about 1904. This type, taken over by Wurl, became known as the Reinsch-Wurl screen, gained in favor and a large number were installed in various cities in Germany. England took up the fine screen idea somewhat later, on individual lines, and we find that the initial installation was made at Carshalton by the John Smith Co. This was a travelling belt, cleaned by brushes and driven by a water wheel operated by the sewage. Numerous other types are recorded, such as the rope screen installed at Manchester about 1923. This consists of an inclined frame with a closely set series of pulleys at each end of the frame and wound over these pulleys ran a continuous wire rope that formed a moving band with spaces approximately ¼ inch or less between the strands. Several of these have been installed in England but due to their cost and the difficulties of repair if one strand of the rope breaks they have not been installed elsewhere.

Fine screening did not progress in the U. S. until several years after the developments abroad. The first screen of strictly American design and use was the Weand, installed at Reading, Pennsylvania, about 1908. This was a revolving drum covered with a fine wire mesh. The sewage entered the interior of the drum and flowed through the meshes. The screenings were carried up on the inside of the drum as it was revolved

and dropped off at the top into a trough hung inside the drum. From there they were carried out through the open end of the screen by a water jet or screw conveyor. Water jets were used to prevent clogging. Other units of this screen were installed at Brockton, Mass., Baltimore, Md., and Atlanta, Georgia. In Baltimore the unit was installed following the sedimentation tanks to remove particles that would tend to clog the nozzles of the trickling filters.

About 1914 Fort installed the first Reinsch-Wurl screen at the experimental sewage plant at Brooklyn, N. Y., and several units were later installed in a permanent plant. In 1915 the first municipal plant using this type was put into operation at Daytona, Florida, under the direction of George W. Fuller. A number of these units have been installed since, mainly in the vicinity of New York City. At present there are about 32 units of this type in the United States.

Fine screening in the U. S. was given consideration mainly for beach protection and at locations where the removal of the larger solids was the main requirement. About the time that interest began to grow in the newly developed "activated sludge" process it was realized that removal of the larger solids was desirable, and consideration was given to the installation of fine screens in the large plant then under study for Milwaukee. About 1916 Tark developed a revolving drum screen. The first unit of this screen, a drum 3 feet in diameter and 3 feet long, was built in a shop in New York City, under the supervision of the author, and when completed was shipped to Chicago. It was later installed at Pleasantville, New Jersey. This unit consisted of a drum covered with perforated metal plates and revolving clockwise. The sewage impinged against the side of the drum and passed through the slots. The solids removed from the sewage were caught on retractable plates, which were carried upward as the screen revolved. Near the top of the cycle longitudinally travelling brushes removed the collected screenings, and the plates were drawn into the interior of the drums by cams, to avoid damage or interference with the brushes. Eight units of this type were installed in the Milwaukee plant. There are now 17 screens of this type in the United States. At Pleasantville, N. J., a second unit was installed, which had, instead of perforated plates, a surface made up of tightly stretched wires. As it was found impossible to prevent objects being forced between the wires the medium was not effective for the purpose.

Development along the line continued, however, and about 1917 a drum screen was developed which operated on a different principle. In this unit the drum revolved against the direction of sewage flow, the solids were carried down and under the revolving drum and brought up on the upcoming side. Because of the troubles encountered in keeping brush-cleaned fine screens free from clogging and to prevent this

clogging and the wear of plates and brushes, this drum screen was designed to be self-cleaning. This was accomplished by revolving the drum at a speed sufficiently high to cause the formation of an artificial head of water on the uprising interior of the drum. This head of sewage caused a flow outward through the slots, thus washing the solids on the exterior face of the drum off into the screenings pit. By means of a slow moving bucket elevator with perforated buckets the screenings were lifted from this pit, draining in transit, and dumped into cans at the top. The first unit of this type was used in handling glue factory and tannery wastes and proved so satisfactory that in 1920 and 1921 the first units were installed at New Britain, Conn., to handle municipal sewage; the two original screens are still in operation there. Owing to freedom from wear and tear this type has been installed in more plants in the U. S. than any other. There are now 92 units of this type in operation in the U. S. and other countries.

A band screen was installed at the Union Stock Yards in Chicago by Jennings about 1913, for the purpose of removing paunch manure, bits of flesh, entrails, etc., from packinghouse wastes. A similar unit was built in New York City, at the Canal Street plant where three of these band screens are now in operation.

The use of fine screens has been limited during the past few years, due to the fact that a fine screen will remove only those solids larger than its apertures. Removals of from 5 to 15 per cent of the suspended solids are generally recognized as being the average to be expected from fine screens with slots of $\frac{3}{64}$ or $\frac{1}{16}$ inch width. Periodic removals of higher percentages may be accomplished by allowing the screens to blind and form a blanket on the surface, but when this mat is cleaned off the removals drop back to normal.

In practically all cases except for beach protection, where removal of visible solids is sufficient, fine screening has been waning in this country and all of the more modern plants have instead been adopting preliminary sedimentation. The cost of fine screening per part per million of suspended solids removed is very high, both from the standpoint of initial and operating costs, compared to similar costs for one of the modern mechanically cleaned sedimentation tanks. A tank with an average detention period of two hours will normally remove from 50 to 60 per cent of the suspended solids at rarely more than double the cost of the fine screening plant, and in many cases at very little extra cost. The main argument for the fine screen is that it requires a very small area to treat a large volume of sewage and is suitable for installation on sites surrounded by buildings of a residential character.

Grit Removal.—The first grit chambers were long, narrow channels, of slightly greater cross section than the sewer, designed to allow the

heavier particles to settle, but to provide sufficient velocity to transport the lighter organic solids. It was necessary to provide duplicate channels so that one or more could be cut out of service during the cleaning period. To clean these channels it was usually necessary to drain off the supernatant sewage and shovel out the accumulation by hand. Owing to the presence of decomposing organic matter, and depending upon the interval between cleanings, this was a noisome job and the resultant "muck" was an unpleasant mess, to be quickly gotten out of sight. The first attempt at mechanizing this procedure was apparently made at Wiesbaden, Germany, about 1900 by Schneppendahl, when he developed a grit chamber with a travelling bucket elevator to remove the grit and to discharge it into cars running alongside at the ground level. Modifications of this type were made, mainly in the form of travelling cranes mounted on tracks over the channels, in some cases using grab buckets or in others depending upon manual shovelling into the buckets and using the crane simply as a means of lifting the buckets. In one or two cases, as at Cleveland, Ohio, the grit chamber bottom was made into a series of hoppers the same size as the bucket, so that each grab of the bucket would clean out a hopper. No attempt to clean the grit removed was made until the installation at the Northeast Works in Philadelphia. This installation consisted of a suction pump on a travelling crane. The pump removed the grit from the three channels and pumped it into a cleaning trough hung on the wall of the building which contained a screw conveyor. After washing, the grit was discharged into storage bins for final disposition.

The nuisance caused by accumulations of grit containing high percentages of organic matter proved that there was a need for a grit collecting unit, which would not only remove the grit from the tank but also free it from organic matter. A unit which had been used successfully for some time in the metallurgical industry for making a separation between particles of different specific gravities was installed in 1926 at the Northeast Works in Philadelphia. This unit proved in a very short time that it would deliver a clean, dry grit directly into cans, carts or other receptacles, without auxiliary handling. Following the demonstration at Philadelphia a unit was developed suitable for municipal installation and installed at Harrison, N. Y. This delivered grit of such freedom from organic matter that it was piled around the building and still remains there. This unit was in continuous service until the plant was abandoned due to the incorporation of the sewer system in one of the large Westchester County projects.

The essential features of this so-called detritor are:

1. A square tank, designed to provide a velocity of not over one foot per second for the maximum flow.

2. A flat bottom, with a maximum depth only 6 inches deeper than the invert of the inlet sewer.
3. A mechanism, revolving in the collecting tank, carrying the solids to the periphery, where they drop into a sloping aperture which leads to the cleaning channel, located directly alongside the collecting tank.
4. A mechanism in the cleaning channel with draining, which carries the mass of solids up the sloping channel, and allows them to drain free of water. This channel is sloped at 2½ inches to the foot. During the alternating motion of the rakes a wave action is caused on the upward raking stroke which floats the lighter suspended matter back into the collecting tank, assisted by a small fast-moving impeller.

Other types of grit collection and washing consist primarily of the conventional long narrow channel equipped with a drag conveyor to collect the grit and a subsidiary conveyor or screw dragging the material up the inclined end of the channel to a point above the water line to provide the washing. One of the later ideas developed for grit collection has been a unit consisting of a series of concentric baffles in a tank, the sewage flowing between and up and over these baffles, the heavy material falling to a cone shaped bottom.

Sedimentation.—The fourth and next step which received attention was the settling tank. The early forms of sedimentation units were simply rectangular concrete boxes, with sloping bottoms, or a series of hoppers, to allow the solids to settle and be removed by dewatering the tanks and flushing out the sludge or manually sweeping it with hand propelled squeegees. The tanks had to be taken out of operation during the cleaning operation. Accumulations of organic solids produced odorous surface scums due to septic action, and ebullition of gas-buoyed particles. The accumulation of sludge also shortened the settling period. The shallower these tanks were the more serious this difficulty became. Troubles with the earlier septic tanks, hydrolytic tanks, etc., led to the development of two-story tanks by Travis and Imhoff in which the sludge was kept in a trapped lower compartment where it could not interfere with the settling of solids. These sludge compartments also permitted the sludge to decompose under a cover of water to prevent odors.

The first recorded attempt to adopt a mechanical means for the collection of sludge in a sedimentation tank was apparently made about 1908 at Bolton, England, and was known as the Ashton Mechanical Squeegee. This was simply a broad scraper board extending across the tank and supported on light tracks running longitudinally at the bottom. When it was desired to collect sludge the supernatant liquid

was drained off, the squeegee placed at one end and a flow of clarified sewage admitted behind it, the pressure of this confined head of liquid forcing the board ahead and pushing the sludge in front of it. It was reversed in a similar manner. The basic fault of this unit was that it required putting the tank out of service during the de-sludging operation and thus required duplicate units, increasing construction cost. There is record of several installations in England of curved blade scrapers revolved by hand, as developed by Pfidler, but none of these came into general use.

Fuller in 1912 says, in referring to "Sludge Removal Devices," "In America there has been no experience with mechanical appliances such as dredges or mechanically driven scrapers to push the sludge to a sump. Arrangements of this sort are to be found in Europe at some of the larger plants."

This statement may have been a challenge to American ingenuity and inventive ability, for we find that about 1913, one of the leading designers of sewage plants in this country, faced with the problem of designing a very large plant, conceived the idea that a mechanical sludge collecting device would be of considerable value. A cleaner developed as a mechanical unit for metallurgical work seemed to meet the requirements. It consisted of a central shaft, with two or more radial arms equipped with a series of plows or rakes, set at an angle of about 30 degrees with the arm, which collected the settled solids and carried them to a sump at the center of the tank. The first unit of this type was one used experimentally in the Stockyards in Chicago in a plant to treat packinghouse wastes. Later, an experimental unit was included in the test plant at Milwaukee, in 1916. These two units were the inception of a development in sludge collecting and sedimentation tank design that has grown to such an extent that today few plants of any size are employing any form of sedimentation unit for any phase of sedimentation that is not mechanically cleaned.

The early units were adaptations of units that had been successful in other fields, but the later ones have been designed regardless of their application to other uses and have been made to meet the severe conditions existing in sewage works operation. Much attention has been given of late to architectural and aesthetic considerations to permit the sewage plant to take its place with other municipal utilities in presenting an attractive appearance.

The early units brought out clearly that this type of sludge collector had many inherent advantages. These, broadly, are:

1. Continuous sludge collection allows the construction of shallow tanks, thus reducing plant construction costs.

2. Fewer tanks are needed, as continuous operation and sludge collection remove the necessity for periodical shutdown and unwatering. Many plants today have one mechanically cleaned tank and have operated successfully for years; in some cases the liquid has not been drained down for 4 to 5 years.
3. Elimination or reduction of surface scums, as removal of solids while still fresh, prevents gas formation and buoyant particles.
4. Removal of sludge daily in volumes equal to the amount deposited maintains maximum volumetric efficiency of tank at all times.
5. Less odors from the sedimentation step, and better appearance of plants, due to absence of scum on the surface of tanks.

The first units were built for central drive with a central shaft and were used in circular tanks. A later type was developed for square tanks. This was still operated by central drive, but with an articulated end on two of the arms, which swept out into the corners of the tank. The use of square tanks introduced several additional factors of economy, namely:

1. Use of common walls between tanks, cutting down concrete quantities.
2. More efficient use of ground area.

The first units also used the truss bridge construction spanning the tanks for supporting the mechanism. A later unit differed from the original in that it was propelled by a motor drive located at the wall end of the truss, or oscillating bridge. This drive carries the carriage around on a rail laid on the tank wall. The entire mechanism turns on a concrete pier in the center of the tank and the truss arm, which extends to the wall, retracts and expands as the drive unit follows the wall of the square tank so that the corners are raked free of sludge as well as the center of the tank.

The latest development is a unit for either circular or square tanks, with a central hollow pier which serves as the sewage inlet. The sewage flows outward to the peripheral overflow weir with constantly decreasing velocity favorable to efficient removal of solids.

For rectangular tanks, a mechanism was developed by W. M. Piatt, of Durham, North Carolina, in 1920, for use at the activated sludge plant designed by him for Gastonia, N. C. This consisted of two long chains supporting between them sludge scrapers of wood. These chains were driven by a motor at the top of the tank and the wooden flights dragged the sludge along the tank bottom to a sump at the inlet end, from which it was lifted by air. Later developments of this type have been the addition of extra sprockets to cause the chain to pass near the surface of the tank on its return journey and use the same flights which

have raked sludge as rakes to convey scum to a trough at the outlet end of the tank.

Another type of mechanically cleaned settling tank, recently developed, consists of a revolving mechanism, driven from a central shaft, but instead of plows to rake the sludge, the arms are provided with nozzles, through which the sludge is removed by means of hydrostatic pressure.

Where sewage is adequately pre-treated by bar screens the scum formation on a primary unit will be small, but in cases where greasy wastes are present light scums may appear on the surface of any sedimentation unit. Such scum is objectionable in appearance and may cause odors and breed flies. It is therefore common practice now to provide a skimming arm on the clarifier unit. The scum is discharged directly into containers and may be burned with screenings or in some cases put into the digestion tanks.

Noting the successful and wide adoption of the mechanically cleaned settling tank in the United States and Canada, the designers of plants in other countries have followed suit until today many plants in England, Germany and other European countries are using the mechanical clarifier. Installations have also been made in Japan, South Africa and Australia.

Sludge Digestion.—One of the first problems encountered in the use of the mechanically cleaned tank was the disposal of the undigested, wet sludge that was delivered daily. In the two-story type of tank this sludge was handled in the same structure, but in the mechanically cleaned unit the sludge must be digested in separate tanks. The early units employed for this purpose were plain tanks, either circular or rectangular, mostly open, and subject to the retarding effect of cold weather on sludge digestion. Owing to the gasification of the digesting solids heavy surface scums were common and were the source of odor and fly nuisances. To determine the value of certain ideas to remedy some of these difficulties an experimental plant was built in 1921 by the Dorr Company, immediately adjacent to the large Imhoff tank plant in the Irondequoit district of Rochester, N. Y. This plant consisted of a small circular clarifier and a circular digestion tank equipped with a mechanism consisting of a central shaft with a set of arms at the bottom and a second set at the top. Operation of this digestion tank continued for 18 months during which time much information was collected concerning the technique of sludge digestion, which led to the design of improved digester units. This early work also inspired much of the research work in this country by Rudolfs, Fair, Downes, Fischer and others, which has developed the basic factors of many of the reactions which are now thoroughly understood and appreciated, and which have put the separate digestion of sludge on the sanitary map very definitely.

The basic theories evolved primarily by the work of Dr. Rudolfs were that the alkalinity or acidity of the digesting sludge made a tremendous difference in the time in which well digested sludge could be obtained, and in the odors and nuisances which accompanied it; and also that the provision of an environment conducive to the rapid action of the digestive bacteria was one of the most potent factors in expediting the action. Tests proved that when sludge was maintained in an alkaline condition at all times, digestion proceeded with rapidity and freedom from odors, foaming, or other troubles common with some of the older forms of tanks. Additions of small amounts of lime were usually sufficient to provide this condition; in a number of cases experience showed that usually the additions of lime need continue for only a few weeks and then the digester would usually function satisfactorily for long periods, sometimes several years, without further additions.

The use of heat in separate digestion tanks was a development more important even than the control of reaction. Temperatures below 40° F. were known to be inhibitive to bacterial action in digesting sludge. At 75° F. or slightly more, the digestion process was found to continue most actively. Consideration of these facts led to research into the possibility of providing the heat in the form of the gas which is the main by-product of the decomposition of organic matter in the alkaline phase. The dependability of the source of supply and the means for collecting and using this gas were the next steps to be worked out. Downes studied this problem at the Plainfield, N. J., plant and in 1925 installed a cover for trapping gases, which were burned to heat water, which heated the sludge in the tank by means of circular pipe coils. The first covered gas collection type of Dorr digester provided with heating coils was designed and installed at Antigo, Wis., in 1926.

A number of installations using the open type of unheated digester were made in the United States between the time of the first municipal plant at Brownsville, Texas, in 1923 and the installation of the heated digester at Antigo in 1926.

The Antigo digester was put into operation in sub-zero weather, with the tank filled with warm water. Digestion started at once and within a few weeks sufficient gas was being produced to permit its use under the boilers. Records show that this plant over a period of 5 years, from 1926 to 1931, produced an average of 1.01 cubic feet per capita per day of gas suitable for burning. A survey of a number of plants of this type, made in 1932, showed that ten of them produced an average of 1.0 cubic foot of gas per capita per day.

The first digester mechanism was driven from a central shaft supported from a truss spanning the tank. In a more recent type the

roof is supported from a grillage of beams which are supported on an umbrella-like structure carried on a central concrete pier.

Another development of heated digestion tank is the Pacific Flush Tank Co. unit designed by Downes of Plainfield, which consists of a steel or wooden floating roof, which rides on the surface of the sludge. This maintains a positive pressure on the sludge gas at all times. As the level of the liquid in the digestion tank rises and falls, the cover follows, and in order to allow freedom of action a free space of 3 to 4 inches must be provided around the entire periphery, open to the atmosphere.

Another development, by the Link Belt Co., is a mechanism for rectangular tanks, similar to that used in the drag conveyor type of clarifier, consisting of chains with wooden flights to agitate the sludge. Several of these have been installed up to date. Several installations of a digester, with domed roof, fostered by The Hardinge Company, have been made. Developments in Europe have paralleled those here and types of digesters divergent in design or equipment but similar in theory have been designed by Pruss and Fries in Germany.

One of the most important features of the use of a completely closed digestion tank is that all odors are controlled and whether the gas is burned for its fuel value or wasted through burners for disposal this adequately eliminates the possibility of odor dissemination. This fact alone has made it possible to build plants on their logical geographical site, namely, that locality to which the sewage will gravitate naturally with the least pumping and with the shortest possible sewer lines.

Sludge Dewatering.—The handling and disposal of the digested sludge has always been one of the major problems of the separate digestion plant. It has been the usual practice to discharge the sludge to open-air sand beds and allow it to drain and air dry to a consistency suitable for fertilizer or fill. Most of the states which provide definite rules for sewage treatment units require that 1 square foot of sludge drying bed area must be provided for each person, based upon designed population. The size of a sludge drying bed, of the open type, for a city of 50,000 people thus amounts to 1.15 acres. Open-air beds are subject to the retarding effect, on drying, of rain and frost. These objectionable features of open beds have led to the adoption of glass covers, similar to those used for greenhouses. Beds thus covered have double the drying capacity of open beds. This increased capacity does not always reduce the total plant cost, as in most localities the open bed may be built for from 30 to 50 cents per square foot, whereas the covered bed usually costs around $1.30 or more per square foot. Economy may come, however, in those cases where large areas of land are expensive and frequent rains retard drying.

As larger plants have been designed and built other means have been

sought to dewater the sludge more rapidly and efficiently. In some of the early plants, especially those where chemical precipitation was employed, filter presses were used to squeeze the sludge to a pasty consistency, making it easy to shovel to barges or carts. It is known that as early as 1855 such presses were used at Leicester, England. In the United States the chemical precipitation plants at Worcester, Mass., and Providence, R. I., made use of sludge presses as early as 1890 for the former and about 1899 for the latter. The sludge press works fairly satisfactorily on a sludge containing a high percentage of lime but is not suitable for digested sludge.

The need for obtaining some means of dewatering the large volumes of sludge discharged from the activated sludge plants led to experiment with the "vacuum filter" at the time the plant for Milwaukee was being designed, the final step in which layout was the preparation of the sludge for fertilizer. The first experiments on this method of dewatering were made about 1921 with an Oliver filter and were so encouraging that when the plant was built a complete battery of 24 large units of this filter were installed. The first municipal installation of this filter for dewatering of sludge was at Pasadena, Cal., in 1925. Other installations were made for filtering activated sludge at Gastonia, N. C., Charlotte and High Point, N. C., Houston, Texas.

The economy of the mechanical, or vacuum filter for dewatering sludge, compared to drying on sand beds, is shown in the table below. It is the author's belief that most of the plants of the future, especially of the larger size, will use mechanical filters for sludge dewatering.

COMPARISON DIFFERENT METHODS OF DRYING DIGESTED SLUDGE
On Basis of 100,000 Population

Type of Drying	Area Required Sq. Feet	Dry Solids Cost per Ton	Remarks
Open sludge beds	100,000	$6.90	Note *A*
Glass covered beds	50,000	9.13	Note *B*
Vacuum filtration	65	6.39	Note *C*

All based on 11.5 tons dry solids per 1000 of population per annum. Sludge from separate digestion tanks. Filter installation capable of handling population of 250,000 by increasing operating time to 20 hours without increase in capital charges.

Note A.—1 sq. ft. per capita. Initial cost $0.50 per sq. ft. Sand replacement $0.15 per ton of sludge handled. Interest, sinking fund and depreciation at 11%.

Note B.—0.5 sq. ft. per capita. Initial cost $1.30 per sq. ft. Sand replacement at $0.15 per ton of sludge handled. Interest, sinking fund and depreciation at 11%.

Note C.—Filter rate 6 lbs. dry solids per sq. ft. filter area per hour, $FeCl_3$ at 6%, or 120 lbs. anhydrous per ton of dry solids. $FeCl_3$ cost at $0.02 per lb. Power, $0.02 per kwh. Filter operation 8 hour basis. Interest, sinking fund and depreciation on building 11% and on equipment 16%.

Power Generation.—Concurrently with the introduction of heated sludge digesters and the use of digester gas, has come the knowledge that in practically every case, more than enough gas is produced to provide heat for tanks and buildings. In the modern type of plant, using mechanical units for the bar screening, grit removal, clarification, digester, and with oxidation by trickling filters, there is usually sufficient gas produced by a given volume of sewage to provide the necessary power to run all of the motors required, excepting only the pumping of the sewage. In activated sludge plants the power can be used for compressing air.

The use of sewage gas for driving engines is not new. At Bombay, India, as early as 1903, gas was used to operate an engine at the Matunga Leper Asylum, which was used to pump sewage. Eddy and Kinnicutt experimented with Imhoff tank gas at Worcester in 1901. The use of this method of power generation has steadily grown in England and Germany. The first engine to run on digester gas was installed at Birmingham, England, in 1927. Others have been added until now a total capacity of 950 H.P. is provided. Many of the modern plants in Germany have large installations of this type, using a horizontal engine of high efficiency, made by Deutz at Cologne. These engines are in use at the largest activated sludge plant in Germany, at Stahnsdorf, Berlin, and also at Leipzig and a number of the other plants which have been built under the jurisdiction of the Ruhrverband and Emschergenossenschaft in the Essen district. Two plants are now under construction in Scotland where gas will be used for power, and several of the newer plants to be built in this country will undoubtedly use it. At Springfield, Ill., a noteworthy installation has been made and one at Rockville Center, Long Island. An installation was made at the Sugaw Creek plant at Charlotte, N. C., but troubles with the engine used have made it an unreliable source of power at that plant. Engines for this type of work must be of a type suitable for burning high temperature gases, as the average B.T.U. of digester gas is 650 per cubic foot, and in some plants even higher. The engine must also be able to run on gas which at times has a high content of hydrogen sulphide, as the use of scrubbers to remove this gas is not general as yet. To smooth out inequalities in supply so that engines may run without interruption, it is usual to provide a gasometer for the digester gas. It has been a noteworthy point, however, that the supply of gas from a modern gas collection type of digester is remarkably uniform in quantity and quality.

Aeration and Activated Sludge.—The most modern step in the oxidation of sewage has been the development of "activated sludge." One of the largest items of cost in the operation of this process has been the cost of power for compression of air. Experiments to reduce the cost of

aeration by reductions in horsepower were first devoted to a departure entirely from the use of added air and a dependence upon sufficient oxygen being absorbed from the atmosphere to oxidize the organic matter. The results of this were the Haworth, Simplex and other mechanical units. In later developments a combination of mechanical stirring by paddle wheels, plus a small amount of compressed air, was used. Imhoff at Essen-Rellinghausen reported that a series of slow moving paddles would keep the solids in motion and that instead of 1.0 or more cubic feet of air per gallon he needed only 0.2 to 0.3 cubic foot per gallon. The combination was investigated in 1929 at the Calumet plant of the Sanitary District of Chicago. Later, the same year, a battery of units was operated at the activated sludge plant at Salem, Ohio. These units were run in parallel with straight diffusion units to ascertain definitely whether the paddle wheel-air diffusion tank would produce an effluent of equal quality in the same time period as the diffusion type. During this work it was found that satisfactory results could be obtained with as little as 0.3 cub. foot of air per gallon. This demonstration indicated that by this combination it would be possible to reduce the horsepower demand about 50 per cent. At the conclusion of the Salem experiments several municipal plants were designed using this type of aerator and to date four of the modern plants in the U. S. are using this system of aeration with satisfactory results and costs. These plants are at Escanaba and Muskegon Heights, Mich., Phoenix, Ariz., and Newark, N. Y.

Other units have been developed for aeration purposes and are in use in this country, such as the Link-Belt screw type with a ribbon paddle operating close to the surface of the sewage; the brush type, developed by Kessener of Holland, which employs a rapidly revolving brush which dips into the sewage and sprays it into the air and keeps it in circulation to provide surface contact with the air. The Simplex unit, similar to its English counterpart, has been used in quite a number of small plants.

Sludge Bed Cleaning.—The customary practice for removal of sludge from sand beds has been by manual shovelling, throwing it into cars, wheelbarrows, etc., and hauling it away. This system has been varied by having tracks laid down the center of the beds, connected to a main line of industrial railway and using gasoline driven locomotives to pull the full and empty cars to and from the dump. In some few plants the use of loaders has been tried, as at Worcester, Mass., but the first large scale machine developed in this country for mechanically removing the sludge from the surface of the bed and delivering it into the cars was the installation made at the Calumet plant at Chicago in 1929 of the unit developed by the Evers-Sauvage Company, of Chicago.

This unit consisted of a truss structure spanning the width of the bed, fitted with wheeled carriages at each end, which ran on rails laid on the bed walls. The machine was driven by an electric motor. Travelling transversely of the bed and mounted on the truss was a bucket conveyor with special buckets designed to pick the sludge up without disturbing the sand. These buckets delivered the sludge to a belt conveyor which carried it to the side of the bed and discharged it into the carts or cars.

This unit was so successful that very large ones were built and installed at the West Side plant in Chicago.

At the Irondequoit plant in Rochester a large travelling crane was provided for depositing buckets on the part of the bed from which sludge was being shovelled and for carrying these buckets to the cars waiting in the depressed alley between the beds. This unit did not remove the sludge from the surface of the beds, depending upon manual shovelling for that.

The units at Chicago are perhaps too large to consider on any but very large plants, with large areas to cover and large quantities of sludge to be removed. A unit of somewhat similar type has been installed at the new Stahnsdorf plant at Berlin.

The basic factor behind the development of all of the units that have been mechanized, is, as will be seen by the foregoing, the relief from some form of trouble or expense, either elimination of manual handling of sewage solids, the cost of multiple units, or better and more efficient use of sewage by-products. This has been carried forward until today it is possible to provide each step with a proven mechanical means of operation, from the mere act of coarse screening down to so-called complete treatment.

One of the early criticisms of the adaptation of mechanical units was the extra cost of the electric power that would be required. This has not proven to be serious, and in fact in many cases the reverse is true, since with more reliable operation less attention has been required and it has been possible to cut down the operating staff.

The trend of thought is toward further mechanization and the concentration of plant units into closely knit groups which may be located in districts previously held inviolate against the invasion of a sewage plant. The advance in knowledge of the principles of sewage treatment, and the disposal of the solids and by-products, has been, of course, greatly responsible for this, but it also is known that mechanization of sewage treatment units has been one of the most important factors in changing sewage treatment from a hit-or-miss hand operation to the present stage of mechanical efficiency.

OPERATION OF THE SEWAGE TREATMENT PLANT OF THE DECATUR SANITARY DISTRICT, 1928–1933

By W. D. Hatfield

Superintendent

Numerous reports on special features of operation of the Decatur sewage treatment plant have been made during the past ten years. No annual reports have been made, although very complete laboratory, operation and audit records are kept, from which average data on daily, monthly or annual operations and costs may be obtained.

The first four years of operation, 1924 through 1927, were years when the capacity of the plant was greatly exceeded due to the organic strength of the starch waste from the Staley Manufacturing Company. A rather detailed description of this four-year period and the work accomplished are contained in the paper by Mr. S. A. Greeley and myself entitled "The Sewage Disposal Works at Decatur, Illinois" published in Proc. Am. Soc. Civil Engs., Oct. 1928, Pages 2238–2286.

A six-year period of operation of the new pre-aeration plant has now been completed and it seems a logical time to make a report summarizing the operation during the period including the years 1928 through 1933.

New Construction

The new pre-aeration plant was placed in operation December 31, 1927 and by May of 1928 the strength of the starch waste was reduced by the Staley Company so that the newly enlarged disposal plant was able, for the first time, to treat all of the city sewage and wastes except at times of high water in the river, when the Imhoff effluent had to be bypassed directly to the river.

Pre-aeration Bypass.—In the summer of 1928 a bypass was constructed which allowed the operation of the pre-aeration plant during high water in the river so that at such times the settled aeration effluent instead of the Imhoff effluent could be bypassed to the river.

Secondary Tank Sludge Line.—The sludge from the secondary humus settling tank had been disposed of by lagooning in low ground just south of the dike which surrounds the plant. This procedure was unsatisfactory and experiments showed that the sludge would be beneficial in the pre-aeration plant. In the summer of 1929 a pressure main was connected from the secondary sludge pump to the wet well of the pre-aeration plant. Since that time this sludge has been returned to the pre-aeration plant, and eventually back to the influent of the Imhoff tanks with excess pre-aeration sludge for digestion.

Effluent Pumps.—In the summer of 1930 the effluent pump house was completed. These pumps allow the operation of the sprinkling filters continuously, regardless of high water. Before the pumps were installed the filters were out of operation about 25 per cent of the time. This construction was done as a protection to the water supply of Springfield, Ill., which lies about 60 miles downstream.

Storm Water Treatment.—In 1932 additions which had been contemplated for four or five years were made to help out the unemployment situation. These improvements consisted in remodeling the grit chambers to simplify their operation, building a three-flight Link-Belt settling tank for storm water sedimentation, a 50 ft. heated digestion tank equipped with floating cover for digestion of storm water sludge and excess Imhoff tank sludge, a 50 ft. sludge storage tank and pump house for controlling the sludge operations.

The approximate costs of these major construction items, including contract price, engineering supervision, plans, specifications, etc., were as follows:

Bypass, sludge line, etc.	$ 2,159
Effluent pump house	35,243
Storm water treatment and digestion plant	60,141
Total cost since 1927	$ 97,543
Total cost before 1927 for plant	982,223
Total cost before 1927 for interceptor	684,560
Grand total	$1,764,326

Landscaping.—The District has never gone into extensive landscaping of the property at the disposal plant. Although such a plan was prepared in 1924 by Mr. Peterson, it was felt that all money should go into sewage treatment until the sewage treatment problem had been completely solved. On the other hand this important work has not been altogether neglected. Some roadways have been made about the plant, the grass has been kept cut and the trees have been trimmed so that the general appearance of the plant has always been presentable. In 1929 the Park Board planted 35 ash trees along the north dike, and 40 or 50 shrubs along the interceptor and grit chamber at the entrance to the plant.

In 1930 shrubs and flowers were planted about the grit chamber, the office building, the Imhoff tanks, and the blower house. During 1931 and 1932 additional flower beds were installed, but no more shrubs were set out. Numerous trees from about the plant have been replanted around the sludge-drying beds so that eventually they will present a much better appearance.

In the spring of 1931 the Park Board planted 30 three-year-old Lombard poplars about the plant. They also slipped 400 young trees for us in their greenhouse. These were placed around the sprinkling filters and the aeration plant in 1932 and 1933 to make a high wall of trees that will asist in holding back any odors from these units. More of these trees will be set out in the spring of 1934.

During the past few years the weeds and underbrush have been gradually cleared from the area between the south dike and the river, thus beautifying the area about the point where the final effluent flows into the river.

The area east of the Imhoff tanks toward the east dike has been the hardest part of the plant to beautify. With the completion of the storm-water treatment units and the use of relief labor in 1933 this area has been leveled, seeded in grass and planted in shrubs.

The District has taken advantage of the State and Federal relief labor and since November 20, 1933 of the C.W.A. labor. Up to April 1, 1934 we have used, through these different agencies $18,261.85 worth of labor. The quality of this labor has been quite good and a considerable improvement has been made in the grounds, roadways and appearance of the plant.

Plant Operation

River Elevation and Sewage Flow.—Because the elevation of the water in the river and the volume of sewage flow follow each other closely these data are plotted together.

The years 1928–29 were normal with regard to rainfall, while 1930–32 represent drought years. In Figs. 1 and 2 the storm water bypassed at the plant before August, 1932, is shown by a straight line extending up from the normal sewage flow curve. Since August, 1932, this line indicates the amount of storm water treated.

The average annual and monthly maximum and minimum river elevation are shown in Table I, also the average daily sewage flow treated, the amount of storm water bypassed, and the percentage of the year that the sprinkling filters were in operation.

The decrease in sewage flow in 1931–33 was due to the drought and the business depression, especially to more frequent shutdowns of the Staley Company. The normal dry weather flow for the city when the Staley Company is shut down is about 5 m.g.d. and the usual waste from the Staley factory is about 4 m.g.d. Combined flows greater than 9 m.g.d. are due to ground water infiltration or rains and melting snow.

Screenings and Grit.—The coarse bar screen at the inlet end of the grit chamber removes the rags, paper, sticks and other coarse material. The grit chamber removes grit, sand, coal and ashes which are washed

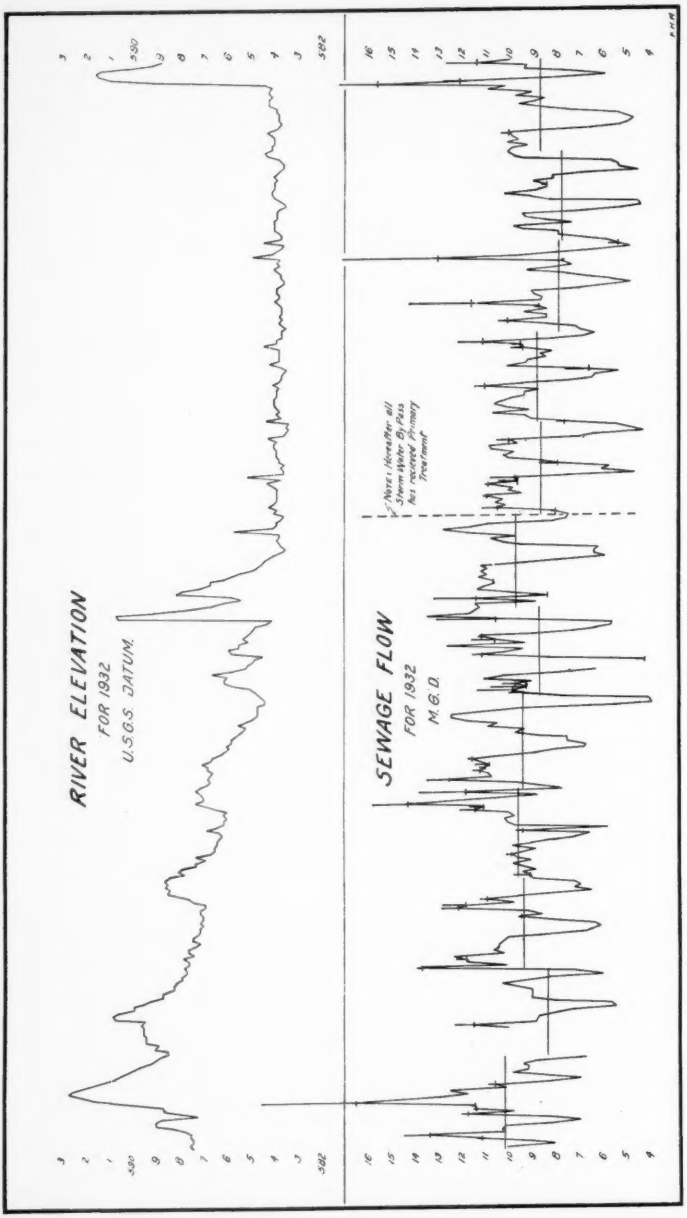
RIVER ELEVATION
FOR 1932
U.S.G.S. DATUM.
SEWAGE FLOW
FOR 1932
M.G.D.
Note: Hereafter all Storm Water By-Pass has received Primary Treatment

FIG. 1.

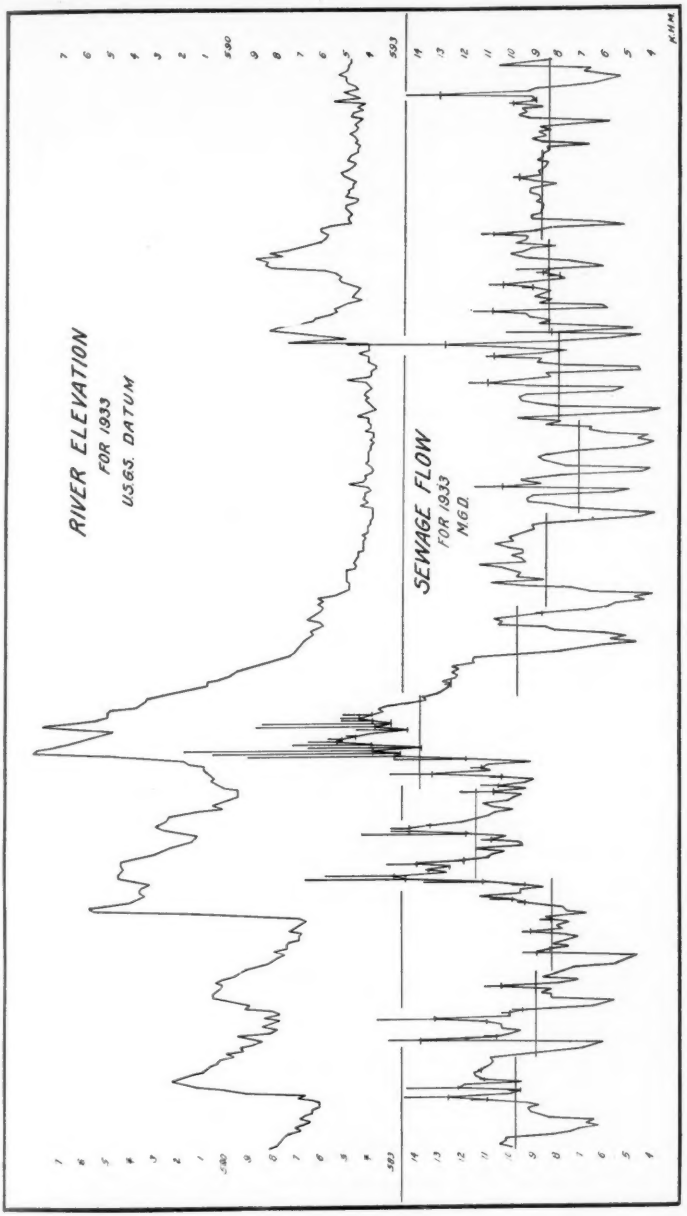
RIVER ELEVATION
FOR 1933
USGS. DATUM
SEWAGE FLOW
FOR 1933
M.G.D.
KHM

FIG. 2.

TABLE I.—*River Elevation and Sewage Flow*

Year	River Elevation			Sewage Flow			Per Cent of Time Sp. Filter in Operation
	Annual Average	Maximum Day	Minimum Day	M.G.D. Treated (Av.)	M G.D. Storm Bypass	M.G.D. Storm Treated	
1	2	3	4	5	6	7	8
1928	588.0	593.8	583.4	10.35	0.54	0	76.5
1929	588.0	596.3	583.8	11.55	.62	0	63.5
1930	588.0	594.7	583.3	10.45	.25	0	77.0
1931	585.1	590.3	583.3	9.00	.12	0	100.0
1932	586.0	592.0	583.4	8.68	.17	0.05	97.0*
1933	587.5	598.1	583.7	9.20	.00	.29	100.0

* The plant was bypassed for 12 days in 1932 due to reconstruction of the grit chamber for the storm-water treatment addition.

from the streets. Table II contains average data on the quantities of these materials removed each year.

TABLE II.—*Removal of Screenings and Grit*

Year	Screenings			Grit		
	Total Cu. Ft.	Cu. Ft. Per Day	Cu. Ft. Per M.G.	Total Cu. Ft.	Cu. Ft. Per Day	Cu. Ft. Per M.G.
1928	1680	4.60	0.444	9,850	27.0	2.61
1929	2070	5.67	.491	12,820	35.1	3.04
1930	1380	3.78	.362	9,850	27.0	2.61
1931	1085	2.97	.330	9,450	25.9	2.88
1932	1380	3.78	.435	8,170	22.4	2.59
1933	1480	4.05	.440	23,600	64.7	7.03

The screenings have been buried and covered with 6 to 8 inches of grit just south and east of the grit chamber. This method has been entirely satisfactory to date and space will be available for a number of years. Grit is being used for sanding the drying beds. In the future some other method of grit disposal may be necessary.

The large quantities of grit received in 1933, must have been due to accumulations in the sewers during the drought years or to the poor conditions of some of the city sewers, for example the Cantrall Sewer which is badly broken in places.

Imhoff Tanks.—The Imhoff tanks have operated quite satisfactorily during these six years. A summary of operating results is given in Table III.

The suspended solids in the raw sewage averaged 201 p.p.m. and in the effluent 85 p.p.m. However, the tanks actually accomplished more during 1930–33 than is shown in the table because the sludge which is pumped back into the influent sewage from the pre-aeration and secondary tanks is not included in the 201 p.p.m. suspended solid in the city sewage, or in the efficiency figure given above. During these three years an average of 2000 to 3000 lb. per day, on a dry basis, of excess aeration and secondary tank sludge was pumped into the influent to the Imhoff tanks.

The amount of sludge in cu. yd. added to the digestion compartment is calculated on the basis of 90 per cent moisture and a specific gravity of 1.03. The solids added per day were from 4 to 7 tons. The rate of accumulation of sludge in the digestion compartment was 1.0 ft. (vertically) in 7 to 12 days, or from 50 to 80 cu. yd. per day. From 60 to 90 per cent of the settled sludge has been drawn to the sludge beds as digested sludge. This sludge dries to about 35 to 40 per cent of its wet volume. Line 12 in Table III indicates the amount of dry sludge removed from the beds (daily average for the year) to the sludge storage pile. During 1929 and 1930 most of the sludge was removed from the beds by the Decatur and Sunnyside Country Club and individual haulers. However, the depression had cut in heavily on this method of disposal and in 1932 this material was piled inside the dike. The available space rapidly disappeared and in 1933 the space outside and west of the dike has been used.

The average daily gas produced is shown in lines 8 and 9 of Table III. An increase in the solids load in the digestion chamber resulted in a decrease in the digestion time and a decrease in gas produced. The increased digestion capacity due to the new separate sludge digestion tank increased the average daily gas rate in 1932–1933 to 55,000 cu. ft. This gas, if used in a gas engine, should develop about 163 horsepower. The average H.P. requirements of our blowers are 167 H.P. with an addition of 15 to 20 H.P. for small pumps, etc. It would seem that we are approaching the possibility of an economic use of our gas for the pre-aeration plant.

The temperature in the digestion compartment of the Imhoff tanks is almost ideal the year around due to the hot starch waste which keep the sewage warm. The summer temperatures in the digestion compartment are between 80° to 100° F. and the minimum winter temperatures are seldom below 65° except when the starch works are closed down for a few days or when the sewage is cold due to cold rains or

TABLE III.—*Operation of Imhoff Tanks*

Year	1928	1929	1930	1931	1932	1933
Average Daily Flow, M.G.D. (1)	10.35	11.55	10.45	9.00	8.68	9.20
Detention Period, Hr. (2)	1.11	1.00	1.10	1.28	1.33	1.25
Suspended Solids, Raw Sewage, P.P.M. (3)	202.	183.	200.	224.	192.	205.
Removal of Susp. Sol., Per Ct. (4)	56.5%	46.3%	56.5%	59.8%	62.5%	62.0%
B.O.D., Raw Sewage, P.P.M. (5)	324.	196.	154.	173.	208.	206.
Removal of B.O.D., Per Ct. (6)	22.0%	21.0%	26.0%	29.0%	27.0%	25.6%
Susp. Sol. Removed Lb. per 24 Hr. (7)	9800	8700	12,400	14,000	11,000	11,100
Gas Produced, Cu. Ft./24 Hr. (8)	50,000	49,300	46,888	46,600	55,000	54,500
Gas, Cu. Ft. per Lb. Susp. Sol. (9)	5.1	5.7	3.8	3.3	5.0	4.9
Susp. Sol. to Digestion Comp. *Cu. Yd. per 24 Hr. (10)	56.5	51.6	71.3	80.5	63.3	64.0
Sludge Drawn, Cu. Yd. (11)	17.3	39.4	66.2	59.5	39.6	51.3†
Dry Sludge, Cu. Yd. Removed per Day, by District Employees (12)	9.5	3.9	2.0	6.25	10.5	23.8
Temperature of Digestion Compartments, F°.						
Summer, Max. Mo. (13)	91	89	93	87	85	?
Winter, Min. Mo. (14)	62	63	59	60	55	58
Average (15)	77	74	74	73	70	?

* At 90 Per Ct. Moisture.

† 1 ft rise in 6 Imhoff tanks equal 16,000 Cu. Ft., or 593 Cu. Yd.

melting snow. The temperatures in Table III are average monthly results recorded daily by a maximum and a minimum thermometer.

Pre-aeration Plant.—The pre-aeration plant has been in operation since December 31, 1927, and the data on its operation are summarized in Table IV. Except for the year 1931 the total flow of the sewage has not passed through the pre-aeration plant, for the following reasons: (1) From January to May, 1928, the waste recovery at the starch works had not become effective and only part of the sewage could be treated, (2) In 1929 and 1930 storm water flows were higher than the hydraulic capacity of the plant, thus making it necessary to bypass some settled sewage to the river when rates of flow became greater than 17 m.g.d., (3) during 1932 and 1933 frequent shut-downs at the Staley Plant made it possible to close down the pre-aeration plant for 5 or 6 days at a time during many months, thus saving considerable power. During these shut-downs all the sewage was passed through the sprinkling filters with success, so that the river was not harmed by such operation and the District saved in power costs at a rate of about $40 a day. A

TABLE IV.—*Pre-aeration Plant*

Year	1928	1929	1930	1931	1932	1933
Sewage Treated, M.G.D. (1)	7.95	10.40	10.22	9.00	7.87	6.74
Per Cent of Total Sewage Flow (2)	76.7	90.0	97.8	100.0	90.6	73.2
Sludge Return, Per Ct. of Sewage Flow (3)	10.5	6.0	4.7	4.3	1.8	2.9
Air, Million Cu. Ft. per Day (4)	4.89	3.98	3.94	4.01	2.57	2.31
Air, Cu. Ft. per Gallon (5)	0.62	0.38	0.38	0.45	0.33	0.34
Aeration Period, Hr. (6)	3.5	2.2	2.0	2.5	2.3	2.6
Total Power, K.W.H. per 24 Hr. (7)	3180	2940	3160	3070	2355	2230
Blowers, K.W.H. per 24 Hr. (8)	2982	2781	2961	2879	2180	1911
Total Power, K.W.H. per M.G. Treated (9)	400	283	309	341	295	331
Total Power, K.W.H. per M.G. Total Flow (10)	308	255	284	341	271	243
B.O.D., P.P.M. in Influent (11)	256	152	111	124	151	153
B.O.D., P.P.M. in Effluent (12)	139	110	75	81	125	122
Per Ct. Removal by Pre-aeration (13)	45.7	27.6	32.5	34.7	17.1	20.1
Susp. Solids, P.P.M. in Influent (14)	88	98	87	90	72	78
Susp. Sol., P.P.M. in Effluent (15)	116	90	85	91	77	73
P.P.M. Removal by Pre-aeration (16)	−28	8	2	−1	−5	5

further saving was made during the spring floods in 1933 by shutting off the pre-aeration plant and running the settled sewage directly to the sprinkling filters. This was made possible because at flood periods the ground water dilution sufficiently reduces the B.O.D. concentration of the sewage so that the filters can handle it without pre-aeration. At such times the effluent pumps are in operation and the net saving instead of being $40 a day, is only about $25 to $30 because the cost of pumping the effluent into the river varies between $8 and $15 per day.

The pre-aeration plant is operated with about 0.33 cu. ft. of air per gallon of sewage, and from 2 to 2.5 hours aeration. This operation has given a reduction of the influent B.O.D. varying from 17 to 35 per cent. The power consumption has been from 250 to 350 K.W.H. per M.G.D. The power costs will be discussed separately later. Suspended matter is formed in the process and although sludge is removed there is little if any improvement in the quantity of suspended matter leaving the process, over that which enters.

Suspended solids are usually slightly higher in the effluent than in the influent because of the "bulky" nature of the pre-aeration sludge. However considerable sludge is formed in the pre-aeration tanks and settled out in the clarifiers. The aeration liquor contains between 150 and 300 p.p.m. of suspended solids and the settled effluent about 80 p.p.m.

In general, the B.O.D. in the effluent from the pre-aeration process (which is dosed on the sprinkling filters) has been reduced between 40 to 50 per cent of that in the crude sewage. The Imhoff tanks reduce the B.O.D. about 25 per cent and the pre-aeration treatment increases the overall removal to from 40 to 50 per cent.

Sprinkling Filter Operation.—There are 3 acres of sprinkling filters, 6 ft. deep, filled with crushed limestone which passes a 2 in. screen but is retained on a 1 in. screen. During the first two or three years, a few of the surface stone split or turned chalky during the winter, but this condition was not bad and has not been noticeable for the past seven years. The surface of the stone is at elevation 595 and the elevation of the river fluctuates between 584 and 600. This allows the plant to operate by gravity about 9 months per year. The effluent pumping station was completed in 1930. Since that time 100 per cent of the sewage flow passing through the Imhoff tanks has been treated by the filters.

TABLE V.—*Sprinkling Filter Operation*

Year	1928	1929	1930	1931	1932	1933
Influent, M.G.D.	10.10	9.65	10.00	9.00	8.68	9.20
Per Ct. of Imhoff Eff. Treated	73.9	59.4	73.0	100.	100.	100.
Days of Operation	279	232	281	365	353*	365
Mil. Gal. per Acre per Day	3.37	3.22	3.33	3.00	2.9	3.07
B.O.D. Applied, P.P.M.	139	110	75	81	125	122
B.O.D. per Acre, Lb. per Day	3900	2950	2080	2030	3020	3120
Pop. Equiv. per Acre	25,200	19,500	13,500	13,150	19,600	20,200
B.O.D. in Effluent, P.P.M.	43	29	20	22	21	24
Per Ct. B.O.D. Removed	69.0	72.6	73.3	72.8	83.2	80.3
Nitrate N, P.P.M.	4.7	2.3	3.8	4.3	5.2	3.2
Stability, Per Ct.	76	68	84	97	98.5	95.0

* The plant was bypassed for 10 days in 1932 for reconstruction and improvements in the grit chambers.

The data on the operation of the sprinkling filters are found in Table V. These results show a rate of application between 3.0 to 3.3 mil. gal. per acre per day, a 5-day B.O.D. loading of 2000 to 4000 lb. per acre per day, or a population equivalent loading between 13,000 and 25,000 per acre per day; a removal of influent B.O.D. between 69 and 83 per cent; B.O.D. in final effluent slightly above 20 p.p.m.; nitrates between 2 and 5 p.p.m.; and annual average stabilities between 76 and 98.5 per cent.

Pooling of the filters due to excessive growth of filamentous organisms on the surface stone has not been a serious factor after the pre-

aeration plant was placed in operation, until the winter of 1932 and 1933. This pooling has been easily handled by picking the surface stone with a pickax. The cause of this pooling is attributed to the fact that during these years irregular operation of the starch industry has made it possible to save power by closing down the pre-aeration plant and passing the domestic sewage directly to the sprinkling filters.

The growth of *psychoda* has never presented a problem at the Decatur plant.

Secondary Sedimentation.—The effluent from the sprinkling filters is settled in one 75 ft. circular Dorr clarifier which was designed and constructed for the original 3 acres of sprinkling filters. The pre-aeration unit allows the treatment of 10 to 12 m.g.d. on these 3 acres of filters and consequently the secondary settling unit is greatly overloaded. The theoretical detention period averages between 30 to 35 minutes at rates of 2000 to 2300 gallons per square foot per 24 hours. In 1931–33 with improved inlet velocities a removal of 25 to 30 per cent of the influent suspended matter and 20 to 25 per cent of the B.O.D. was obtained. Further improvement in distribution of inlet velocities will increase the removals considerably, but the one tank will always be overloaded at the present rate of flow. The average annal data on the operation of this unit are found in Table VI.

TABLE VI.—*Secondary Tank Operation*

Year	1928	1929	1930	1931	1932	1933
Days Operation	110	52	199	365	353	282
Sewage Flow, M.G.D.	10.10	9.65	10.00	9.00	8.68	9.20
Detention Period, Minutes	29.3	30.7	29.6	32.9	34.1	32.2
Gal. per Sq. Ft. per 24 Hr.	2280	2180	2260	2040	1965	2080
Suspended Solids, P.P.M.						
Influent	76	77	55	60	51	64
Effluent	62	62	45	40	38	45
Per Ct. Removed	18.5	19.5	18.0	33.3	25.5	29.5
B.O.D., P.P.M. Influent	43	29	20	22	21	24
Effluent	21	16	15	16	17	19
Per Ct. Removed	51.0	44.8	25.0	27.	19.	21.
Overall Removal, Raw Sewage to Final Effluent						
Suspended Solids (Per Ct.)	68.3	72.	77.5	82.1	80.2	78.0
B.O.D., Per Ct.	93.5	92.0	90.3	90.7	91.8	90.8

Final Effluent.—During 1931 and through 1933 the final effluent from this tank, which flows continuously into the river, contained an annual average suspended solids content of about 40 p.p.m., less than

20 p.p.m. B.O.D., 3 to 5 p.p.m. of nitrates, and 90 to 99 per cent stability toward methylene blue. An excellent testimonial to the quality of the final effluent was the condition of the Sangamon River between Decatur and Springfield during the drought years of 1930 through 1932. For many months at a time no water for dilution of the final effluent flowed over the Decatur water supply dam. About 3 m.g.d. of ground water seeped into the river under the dam. This 3 m.g.d. of dilution water plus the 9 m.g.d. of final plant effluent made up the total flow in the river 15 miles below the plant and no major dilution entered the river until about 37 miles below Decatur. Even under these conditions the river water was clear, odorless and sparkling. The bottom, where rocky or sandy, was clearly visible and free from abnormal growths. No abnormal growths of water weeds has been noticed in the river or along the banks.

Below the sewage treatment plant nine varieties of fish were caught in a seine and subjected to a careful examination. They were found to be normal, healthy, and free from parasites, and the contents of their stomachs and intestines were characteristic of their species. In 1929 the State Natural History Survey made a fish count (unpublished) and found many more varieties and about seven times as many fish a hundred yards below the point of discharge from the sewage plant as anywhere else in the river.

Population Equivalent Balance

For the purpose of presenting the strength of the crude sewage, the efficiencies of the treatment processes and the strength of the final effluent, a population equivalent balance has been used. The relative strength of sewage and effluents is easily visualized by both layman and engineer when expressed in population equivalents. The population equivalent is much more descriptive and equally as valuable a measure, as the "quantity units" of the U. S. Public Health Service and the common "thousand pounds per day applied or removed." The term is especially valuable in comparing industrial waste pollution with that of domestic sewage. This equivalent is based on the determination of the biochemical oxygen demand (B.O.D.) of the samples and of the amount removed by each process. The following equation is used for the calculations.

$$\text{Population Equivalent} = \text{P.p.m. B.O.D. (5-day)} \times \text{M.G.D.} \times (8.33/.17)$$

The population equivalent is calculated each day for the crude sewage, the final plant effluent and any bypassed sewage or partially treated effluents. The population equivalent removed by each process is calculated and the balance struck. In this way we have a weighted average

of the efficiency of each process made up daily and averaged at the end of the month. It should be noted that if for any reason a part of the process is bypassed for a while, its efficiency during the bypassing period is zero and must be averaged in as such for the monthly averages. Further, the population equivalent of the crude sewage is taken as the total received at the plant, and storm water bypassed without treatment is zero treatment, thus a plant bypassing 50 per cent of its sewage at storm times would have a maximum efficiency of only 50 per cent if purification of the remaining sewage were perfect (a hypothetical possibility).

Table VII contains population equivalent data expressed as yearly averages of the weighted monthly averages. The crude sewage shows a drop in strength from a population equivalent of 174,000 in 1928 to

TABLE VII.—*Population Equivalent and Plant Efficiencies Based on Monthly Averages of Weighted Daily Data*

Year	1928	1929	1930	1931	1932	1933
Population Equivalent of Crude Sewage	174,333	111,750	77,500	75,000	92,500	90,400
Population Equivalent Removed by						
Imhoff Tanks	35,800	23,400	19,500	21,333	24,200	22,500
Pre-aeration Tanks	31,000	20,700	17,700	18,000	12,600	13,000
Sprinkling Filters	36,000	28,000	21,100	26,300	43,400	42,900
Secondary Sedimentation	630	160	750	2,363	1,689	1,595
Storm Treatment Tanks	0	0	0	0	135	452
Total	103,430	72,260	59,050	77,996	82,024	80,447
Population Equivalent to River						
in Final Effluent	59,500	33,000	18,700	6,800	7,400	9,900
Bypassed as Storm Water	11,400	6,500	1,080	1,300	3,600	0
Total to River	70,900	39,500	19,780	8,100	11,000	9,900
Per Cent Removal by						
Imhoff Tanks	20.5	21.0	25.2	28.4	26.2	24.9
Pre-aeration	17.8	18.6	22.8	24.0	13.6	14.4
Sprinkling Filters	20.7	25.0	27.2	35.0	47.0	47.5
Secondary Sedimentation	.36	.01	.9	3.2	1.8	1.8
Storm Water Treatment	0	0	0	0	.1	.5
Total Removed by Treatment	59.36	64.61	76.1	90.6	88.7	89.1
Total Per Cent in Final Effluent	34.1	29.6	24.1	9.1	8.0	10.9
Total Per Cent Bypassed as Storm Water	6.5	5.8	1.4	1.7	3.9	0
Total to Check	99.96	100.01	101.6	101.4	100.6	100.0

75,000 in 1931 and then back to above 90,000 in 1932–33. The average monthly data contain days when the starch works were shut down and the population equivalents were as low as 40,000; holidays such as the Fourth of July, Labor Day and Christmas when the population equivalent were as low as 20,000; and occasional days when heavy losses of industrial wastes brought the equivalent up to greater than 200,000. However, in general, the normal population equivalent was between 100,000 and 120,000.

Of this amount the Imhoff tanks removed between 19,000 and 35,000 population equivalent, or a range from 20 to 28 per cent; the pre-aeration plant has removed from 12,000 to 31,000 population equivalent or from 13 to 24 per cent; the sprinkling filters have removed from 21,000 to 43,000 population equivalent or from 20 to 47 per cent; the secondary tank removed about 2 to 3 per cent; and the storm water tank to date has removed only about 0.5 per cent. The total annual removals have varied from 59 per cent in 1928 to 90 per cent in 1931, 1932 and 1933. Also in Table VII the population equivalent of the final effluent entering the river is shown. This figure includes the final filter effluent plus all sewage bypassed at any point during treatment. The progressive increase of efficiencies has been due largely to less bypassing to the river following construction of the effluent pump house and the storm water plant. On the other hand the decreasing efficiency shown for the pre-aeration plant is due to the fact that this step in the process has been bypassed frequently and therefore its efficiency penalized in favor of the sprinkling filters.

The seasonal effect on the total plant efficiency in population equivalent removal is shown in Table VIII.

TABLE VIII.—*Monthly Removals of Population Equivalent in Per Cent*

Month	1928	1929	1930	1931	1932	1933
January	40	50	38	87	92	86
February	23	68	37	92	60.8	85
March	46	36	67	87	89	87
April	36	35	71	86	88	85
May	82	43	83	90	92	80
June	85	63	85	91	89	91
July	90	66	95	92	94	92
August	84	93	93	92	92	93
September	90	95	93	93	92	91
October	89	90	95	91	93	90
November	84	87	90	88	92	93
December	74	81	87	90	90	90

In 1928–30 low winter and spring efficiencies were due to storm water and sprinkling filter bypassing. Since 1930 winter and spring efficiencies have been between 86 and 90 per cent while those during summer and fall have been between 90 and 94 per cent. The low figure for February, 1932, was due to a complete bypass of the whole plant for ten days while the grit chambers were being reconstructed to handle the new storm water treatment plant.

IOWA SEWAGE TREATMENT CONFERENCE AND IOWA WASTES DISPOSAL ASSOCIATION

SELECTED PAPERS *

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UTILIZING EXISTING SEWAGE WORKS TO BEST ADVANTAGE

BY A. H. WIETERS

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We hear a great deal about the need for new sewage treatment plants and the methods of financing construction and operation. With the ever increasing demand on the part of the general public for cleaner streams, the results from existing treatment plants are going to be scrutinized more closely. Many towns which have just been "getting by" will find themselves in trouble in the future. To illustrate what I mean, news reports during the past few weeks have come into the office of the State Department of Health of four Iowa municipalities being made defendants in damage suits because of the discharge of poorly treated sewage—not untreated sewage—on adjacent land. Others have been threatened and if past court decisions are to be taken as a criterion, most of the municipalities involved will lose their cases. The number of cases brought to court at this time is unprecedented in Iowa in spite of the fact that I believe the condition of sewage treatment plants in general has never been better. The State Department of Health has also made an unprecedented number of investigations, on receipt of petitions, of pollution caused by untreated or inadequately treated industrial wastes. I interpret this to mean that the general public is daily becoming less tolerant to pollution; that it is refusing to continue in the attitude that stream pollution is a necessary evil.

Failure of a sewage treatment plant to produce satisfactory results may generally be attributed to five causes as follows: faulty design or construction; obsolescence; overloading; industrial waste; faulty operation.

Due to faulty design or faulty construction, some plants have never worked satisfactorily. Fortunately there are but few of the failures in Iowa which can be attributed to this cause. Obviously no engineer

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would design a plant exactly as he did twenty years ago since the science of sewage treatment has advanced greatly in that time. If a plant was designed in accordance with the best engineering practices of twenty years ago, its failure to come up to the standard of present day design cannot be fairly attributed to faulty design. If a plant fails to function due to faulty design, little can be done to correct the condition other than by reconstruction or replacement. Usually the latter is the more economical procedure.

A second cause for the failure of old plants to meet present day requirements is obsolesence. This should not be confused with faulty design, although the net final result may be the same, *i.e.*, an economical correction of the condition may require major reconstruction or replacement. (For example a Cameron type septic tank is obsolete today in that a situation is hardly conceivable where such tank would be designed in preference to a tank of a more modern type, to take care of municipal sewage.) Under favorable conditions septic tanks today are still giving results satisfactory enough that replacement is unwarranted. In other places, however, due to overloading, introduction of industrial waste, hard water, stale sewage, a demand for better quality of effluent, or other cause it would be economical to replace an existing septic tank with a more modern type of tank, in view of the better results that would be obtained and the protection that would be afforded the filters if a cleaner tank effluent could be produced.

Introduction of industrial wastes is responsible for the failure of many sewage treatment plants. Plants originally designed for domestic sewage have had industrial wastes thrust upon them in increasing quantities until the entire plant ceases to function. Many industrial wastes require special types of pre-treatment before they can be discharged into even the latest type of plant designed for domestic wastes.

An example of this type of failure common in Iowa is due to the rise of the butter making industry. In many places when the sewage treatment plant was built there was no creamery or perhaps a very small one. Many of these creameries in smaller places have grown to such proportions that the sewage treatment plant is entirely unable to handle these wastes. This has become one of the serious causes of the failure of sewage treatment works. The remedy is obvious—namely the pretreatment of the industrial wastes so that the city sewage plant can absorb them, or the diversion and separate treatment of the wastes.

The fourth cause for the failure of sewage treatment plants is overloading, which is a common cause in Iowa. Lack of knowledge on the part of the designer as to the behavior of sewage, a tendency towards holding initial costs to a minimum and unpredictable growth of municipalities since the plant was built are the principal reasons for present day overloading.

For instance, there are many Imhoff tanks in the state with sludge digestion capacities grossly inadequate for the population for which the tanks are designed. This is not a reflection on the designing engineer, since he simply followed the recommendation of the inventor of the tank. The temperature was not considered as important a factor in digestion of sludge as it is now known to be and apparently designers lost entire sight of the fact that sewage in German cities maintained a minimum winter temperature of 67° F. or greater whereas in our small Iowa municipalities winter temperatures of the sewage will drop to 40° F.

There are numerous Imhoff tanks in Iowa which are operating efficiently as settling tanks after sludge withdrawal, but due to inadequate sludge digestion capacity this favorable condition cannot be maintained. In several instances the condition has been corrected by continuing the use of the Imhoff tank as a settling tank and building an additional sludge digestion tank to take care of the excess sludge.

Another common example of failure due to overloading is found in intermittent sand filters. Due to the fact that bank run sand is unsuitable for sand filters in most places in Iowa, most of the sand used for this purpose has to be graded and washed, and in many instances shipped considerable distances. This makes this type of filter quite expensive considering the replacements that are necessary. The early designers cannot be accused of guiding the design of sand filters of inadequate areas. In fact most of the earlier sand filters in the United States were designed much more liberally than the later ones. Economic consideration must be given as the reason for the many sand filters in Iowa which are inadequate in area for the designed load.

In a sense sand filters may be considered as being obsolescent in that very few are now being built as compared with other types of treatment. In spite of this trend from intermittent sand filters, it must be said that a sand filter receiving a well clarified tank effluent and one which is not overloaded will still produce an effluent as good in quality or better than that produced by any of the more recently developed types of secondary treatment. High upkeep and operating costs coupled with difficulties in winter operation have been primarily responsible for the trend away from intermittent sand filters, rather than the failure of a sand filter to produce a satisfactory effluent. There are today in Iowa many sand filter plants which are giving very satisfactory service after fifteen or more years of operation.

The fifth and most common cause for failure of a sewage treatment plant to produce satisfactory results is faulty operation. According to the most recent record of the State Department of Health, of the 216 sewage treatment plants, only 10 can be said to be completely useless and most of these represent old type Cameron tanks with no secondary

treatment. Eight were found in which the tanks were operated but the sand filters had fallen in such state of disrepair that no pretense was made to operate the filter units. Thus less than 5 per cent of the plants in the state which are serving less than 2 per cent of the population served by sewage treatment are being habitually by-passed; and in less than 4 per cent of the plants the sand filters have fallen in such state of disrepair that they are no longer used, but in these cases the tanks are still used. These figures would seem to contradict a statement frequently heard that most sewage disposal plants are failures.

The chief difficulty today seems to be the lapses in proper operation which result in intermittent periods of poor results. These lapses are usually caused by the fact that the operator, who generally has multitudinous other duties, must for a time entirely neglect his plant; or due to periodically recurring waves of false economy when no one is regularly assigned to the task of looking after the sewage disposal plant; or to frequent changes in personnel; or to the occurrence of an unusual condition with which the operator has had no previous experience, and hence he is unable to cope with the problem.

I am afraid that many designing engineers in the past have been too prone—in their anxiety to secure a client—to minimize the operation and maintenance requirements. A sewage plant has not yet been devised which will continue functioning indefinitely unless it receives some attention, although results are nothing short of remarkable, in some instances, where plants receive very little attention.

I do not wish to infer that the average small sewage treatment plant requires a great deal of time nor that it must be under continual technical supervision. Any plant requires the attention of an intelligent operator who understands the fundamental principles involved and who has the time to devote at least a few hours a week to the plant, at regular intervals. Such attention will pay big dividends in the long run, as may be witnessed by comparing a plant so attended with some of the wrecks after a few years of neglect. I recently visited a small-town plant consisting of a septic tank and sand filters and found the plant in excellent condition and delivering a crystal clear non-offensive effluent. Upon inquiry I was informed that the total maintenance cost consisted of the replacement of about ten inches of sand on the filters and a little piping in the dosing tank after twelve years of continuous service winter and summer. I also was informed that the operator visited the plant regularly and although he did not spend much time on these visits he forestalled trouble by attending to the routine tasks such as sludge removal, sand raking and scraping when they were needed. There is no reason why this little plant should not serve this community for many more years although it represents a design now obsolescent.

Recently it was reported that the Baltimore, Md. trickling filters had been in operation for twenty-seven continuous years and that the entire structure, including the filtering material, was still in excellent condition and the plant still operating as efficiently as ever. Imagine a fire truck, a pump or other piece of similar equipment in service after twenty-seven years. Even if such equipment is still in good mechanical condition, obsolescence will have relegated it to the junk heap.

Correct design and good construction are of no avail if intelligent operation is not provided. Recognition of this is forcibly demonstrated by a requirement in New Jersey and Maryland, for instance, that all sewage plant operators must be licensed by the State Board of Health; and in Ohio by a requirement that when plans for a sewage disposal plant are submitted to the State Board of Health, with them must be submitted a budget for operating the plant. Similar requirements are found in other states.

The proper operation of a sewage plant need not be costly, in fact in the long run it is far less costly than neglect. It is of course impossible and unnecessary to provide a technically trained operator for a small plant. An intelligent mechanic can readily familiarize himself with the fundamental principles of sewage treatment, the routine operation tasks, and the necessary simple laboratory tests. If an unusual condition arises requiring technical assistance, the trained personnel of the State Department of Health or the engineer who designed the plant are available to give him advice.

I will cite a specific example of a well operated plant. A small city of less than 2000 population has a modern mechanized plant handling domestic wastes with an unusually large amount of creamery wastes. Added to this the city water supply is very highly mineralized making a combination extremely difficult to handle. The plant is under the supervision of a very capable superintendent of utilities who visits the plant daily to see that everything is going all right. The operator visits the plant twice daily, doing the necessary skimming, oiling of machinery, pumping of sludge and other routine tasks and on an average these visits consume about one hour daily. Saturday afternoon is devoted to a general clean up and tasks which require more time. Routine pH tests of sewage and sludge, settling solids tests, stability tests are made and recorded daily as are other pertinent data such as quantity of sewage pumped, electric current consumed, gas produced, etc. Thus a plant with mechanical equipment and receiving a sewage which is extremely difficult to treat is requiring only ten or twelve hours a week of operating labor. Possibly but few towns are so fortunate in having locally available as capable a superintendent as this town, but on the other hand only a few such towns have a sewage as difficult to treat.

If a sewage treatment plant is needed, and I do not know of a case in Iowa where one was built which was not needed, certainly provision for operating and maintaining the plant should be made as it is false economy to neglect a plant. While there are a number of plants in the State which due to faulty design, obsolescence, overloading or change in the nature of the sewage, are not producing results compatible with present day requirements, and where replacement would be the most economical solution, there are many plants where better results could be obtained if a little more attention were paid to operation.

The State Department of Health through its Engineering Division makes routine visits to existing sewage plants in so far as is possible with the limited personnel. The purpose of these visits is not to find fault, but to advise and aid the operator. In addition the Department, again within limitations of a small staff, makes more elaborate investigations upon request of the local officials on occasions when unusual conditions arise, outside the knowledge or experience of the operator. The engineer who designed the plant is also usually glad to advise in such cases where unusual conditions arise.

PRECHLORINATION OF SEWAGE FOR ODOR CONTROL

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When we think of a sewage treatment plant as a workshop for nature's task of stabilizing the organic material found in domestic sewage, it is not incomprehensible that objectionable odors are apt to pollute the surrounding atmosphere. In normal cases, that is, when the sewage contains no excessive amounts of industrial wastes or other materials, the odors are not concentrated to the extent that a public nuisance condition will exist. However, in the few abnormal cases the objectionable odors have become quite a problem and in this paper I will discuss the experiences along a certain line of correction—that of prechlorination.

Objectionable odors resulting from the treatment of ordinary domestic sewage are caused mainly by the formation of hydrogen sulphide gas. This gas is generated by certain bacteria present in raw sewage and which must have oxygen for support of their life. These bacteria have the faculty of attacking the sulphate radical, utilizing the oxygen and liberating hydrogen and sulphur which in combination form hydrogen sulphide.

Whether the concentration of hydrogen sulphide around a treatment plant is sufficient to cause a nuisance must usually be left to the decision

of persons living in the vicinity. A quantitative analysis may, of course, be made, but due to the marked influence of climatic conditions these tests do not indicate the extent of the nuisance.

Enslow explains in an article on odor control by chlorination, that four conditions must occur simultaneously in order that excessive quantities of hydrogen sulphide gas are produced. These are as follows: (1) The presence of sulphates in the carrying water which constitutes the liquid portion of sewage. (2) Hydrogen sulphide producing bacteria, that is those bacteria capable of extracting oxygen from the sulphate radical, thereby liberating hydrogen sulphide. (3) Proper temperature conditions for bacteria multiplication, and (4) Sufficient time for bacterial activity.

It is further pointed out that if any single one of the above factors are eliminated, hydrogen sulphide production can be controlled.

In most of the municipalities in Iowa the public water supply does not contain an abnormal amount of sulphates, hence the first condition of the above is not satisfied. In these municipalities, no serious difficulty is encountered from the standpoint of odor control. There are, however, a few cities and towns in which the water supply is taken from deep wells. These waters contain excessive amounts of sulphates.

The second condition is not difficult to comply with since all domestic sewage seems to contain the aerobic bacteria capable of liberating hydrogen sulphide.

In three, above, proper temperature conditions are easily complied with during summer months when temperatures seemingly come within the range necessary for bacterial multiplication.

Condition four, that of affording sufficient time for bacterial activity, is readily supplied during the passage of the sewage from the home to treatment plant and during the period of detention within the primary settling tank. This time interval is naturally increased if the collecting sewer system is laid on flat grades and if a long outfall sewer line is available.

In the prechlorination process of odor control, bacteria capable of producing hydrogen sulphide gas are eliminated. Chlorine is a powerful germicide and by applying the chlorine to the raw sewage, all existing bacterial life is immediately exterminated. Thus the organic material containing the sulphate radical is permitted to remain intact and as long as the oxygen remains, the hydrogen and sulphur will not combine to form hydrogen sulphide.

It appears important that the chlorine be applied as soon as possible after the sewage is discharged into the collection system because bacterial activity begins immediately. Prechlorination also limits hydrogen sulphide gas production by combining with the hydrogen, forming hydro-

chloric acid, and the sulphur remains as a precipitate. The cloudy appearance of sewage after chlorine has been added and the settleable solids removed is caused by the fine particles of sulphur held in suspension. Due to the larger amount of chlorine necessary for the chemical reaction, however, it is desirable to utilize the chlorine in preventing the production of hydrogen sulphide which necessitates application at the earliest stage of the treatment process or preferably at some point in the outfall sewer. It is of further economical importance that the chlorine dosage be varied in accordance with the fluctuations in flow of sewage through the plant.

During the past year and a half, the writer had the opportunity of conducting an experiment in odor control by prechlorination. In the town where this experiment was carried on, the sewage plant, consisting of an Imhoff tank and trickling filter, is located very near the residential district. The water supply for this community is taken from one of the deepest wells in the state and is exceptionally high in sulphates. The sewage disposal problem is also materially affected by the discharge of creamery wastes.

Soon after the plant was put into operation numerous complaints were made concerning obnoxious odors. After a preliminary investigation, it was decided that prechlorination was desirable. Subsequently a Wallace and Tiernan dry feed chlorinator was set up to apply chlorine at the entrance to the Imhoff tank. This point of application was not strictly satisfactory in view of the flat grades on which the collection system is laid, but was the best that could be obtained under the circumstances.

Before chlorination, the liquid in the Imhoff tank settling compartment and in the dosing tank was of a dense black color and had the typical stale odor of septic sewage. A laboratory test on the raw sewage coming into the plant indicated a sulphate content of 483 parts per million and a temperature of 16° C. These conditions coupled with the long detention period afforded by the settling tank undoubtedly were responsible for the hydrogen sulphide odors.

The chlorinator was set for a dosage of 12 parts per million with the intention of cleaning up existing conditions and reducing the dosage after a few days of operation.

During the course of this experiment it was found that the obnoxious odors arising from this plant could be materially reduced by prechlorination, even though conditions for economical treatment were not at all ideal. In the first place the sewage had become septic to some extent before reaching the plant and before the chlorine could be applied. Furthermore, the mixing of chlorine with the sewage was inadequate due

to the fact that a dry feed machine was in service and proper distribution could not be obtained.

The chlorine dosage was varied between 8 and 18 p.p.m. depending upon local weather conditions, particularly the direction of wind currents. The maintenance of a slight chlorine residual in the tank effluent was attempted but because of wide fluctuations in flow the dosage was more than enough for hydrogen sulphide reduction at certain times, as indicated by a chlorinous odor coming off the liquid as it was dosed on the trickling filter beds.

After a few days of continuous chlorination the settling tank effluent had lost its black color and appeared to be in a fresh condition. Furthermore the turbidity of the effluent had increased, which would indicate the precipitation of flocculent sulphur and the subsequent reduction in hydrogen sulphide.

Due to the lack of time which could be devoted to this experiment, no determinations of hydrogen sulphide concentration were made at the plant. The results of the experiment were judged entirely on decisions of the local people previously affected. The plant operator reported a remarkable reduction in hydrogen sulphide odors in the immediate locality of the plant and town officials reported a substantial decrease in complaints.

I would not say that all objectionable odors were eliminated by this process but I do believe that the odor nuisance was decidedly reduced.

Experiences in other sewerage plants throughout the country have also shown that prechlorination is particularly advantageous in reducing hydrogen sulphide odors. There have been scores of pamphlets and bulletins written on this subject but the consensus seems to be that

1. Hydrogen sulphide gas is the chief cause of odors.
2. This gas is produced in sewage having a high sulphate content and is actuated by flat grades, long outfall sewers and detention periods within the plant.
3. Less than 1 p.p.m. hydrogen sulphide in the tank effluent is unobjectionable.
4. Chlorine prevents hydrogen sulphide odors in two ways; namely, by killing certain bacteria thereby preventing the production of hydrogen sulphide and by changing the existing gas to a non-odorous form.
5. Objectionable odors can be reduced by treating with chlorine in sufficient quantity to maintain a slight residual in the tank effluent.

In treating sewage with chlorine it is necessary to have only a slight residue of chlorine when the fluid is dosed on the filter beds. The cost can be reduced by opening and closing the valves in accordance with the weather conditions, particularly the direction of the air currents. In

most towns using prechlorination, the yearly average would be about 3 parts per million. We used 8 to 18 parts per million in the experiment I worked on but it was not possible to chlorinate until it reached the tank and some reducing action had already taken place.

Discussion

Mr. Rogers: What system did you use in applying the chlorine and what apparatus did you use?

Mr. Houser: We used a Wallace and Tiernan dry feed chlorinator. The chlorine was applied at the inlet of the Imhoff tank. We did not have ideal conditions. The experiment would have been more successful if we had had more adequate means of applying the chlorine to the sewage before it reached the tank.

Mr. Wieters: In discussing Mr. Houser's paper I would like to say that while sulphates must be present in the water before hydrogen sulphide will be formed, apparently the amount of sulphur that is changed to hydrogen sulphide is very small. When you have heavy concentrations of sulphates in the water you are more likely to have difficulty under exactly the same operating conditions. There is very little water in Iowa that is entirely free from sulphates, but evidently there must be an excess of sulphates before hydrogen sulphide is formed.

Another point in this condition is that while the hydrogen sulphide may be objectionably odorous, apparently there is no serious interference with the working of the plant. You may have a plant that smells very bad and yet have a very clear effluent and satisfactory sludge digestion. We have noticed in the separate sludge digestion plant where the gas is burned, a smell of the gas may reveal a high hydrogen sulphide content, but there is apparently normal digestion.

Another point that Mr. Houser mentioned is that while it is possible to chlorinate a sewage and reduce the hydrogen sulphide after it has been formed, it is more economical to prevent the formation of the gas by chlorinating before the sulphates break down. In the particular instance Mr. Houser mentioned we tried to find a place to apply the chlorine before the sewage came to the tank. The sewage came in three or four branches which terminated at the plant so it could not be treated while it was still fresh. In most places it would be possible to apply the chlorine at some place while the sewage was still fresh and a more economical use of chlorine would be indicated.

I would like to raise a point in regard to hydrogen sulphide gas itself. It takes a very small concentration of the gas to produce an objectionable odor. In fact when the hydrogen sulphide gas is present in larger concentrations it is not as unpleasant as when a small amount is present. But it is poisonous, just as hydrocyanic gas and others. There are cases

on record where people have been killed by it. The fact that it does have such an objectionable odor usually indicates its presence. However, when in large concentrations and the odor is not so bad, one is likely to think that there must be only a small amount of gas present. I think it is well to mention this and we should keep in mind that it is a poisonous and dangerous gas. There have been several fatal cases of such poisoning in Iowa. Mr. Houser did not mention his own experience with hydrogen sulphide gas, but he came very close to experiencing hydrogen sulphide poisoning himself a couple of years ago.

Mr. Galligan: Since there must be an excessive amount of sulphates in the water, isn't it possible that some of the trouble comes from other odorous compounds of sulphur?

Mr. Dye: In the meat packing industry it has been the custom for many years to use saltpeter to preserve the red color of the meat. The saltpeter is reduced to nitrites and these are further reduced to even more complex compounds. We have to use a good many tons of nitrates to produce a ton of nitrite. There is probably some analogy between this and the excessive amounts of sulphates necessary to produce the sulphide compound.

Question: Is there any correlation at all between the sulphate content and the iron content?

Mr. Wieters: Apparently there is no correlation, although the point frequently comes up in regard to iron bearing water. There is often discoloration and unpleasant odor but anyone familiar with hydrogen sulphide could not mistake the odor.

APPLICATION OF THE SEWER RENTAL LAW TO NEW CONSTRUCTION

By Howard R. Green

Consulting Engr., Cedar Rapids, Iowa

The Iowa statutes provide four separate methods for financing sewage treatment works. They are considered as a part of the sewage system and are financed under the general sewer laws. The new sewer rental law adopted by the last legislature is an adaptation of the first sewer rental law. These methods are: (1) Special assessments in ten annual equal payments with an interest rate not in excess of 6 per cent. (2) The issuance of sewer bonds of a general nature and retirement of those by millage levy against the real and personal property in the city, over a 20 year period, with an interest rate not exceeding 5 per cent. Six per cent is allowed on certificates based on special assessment but only 5 per cent when bonds are paid by millage levy. (3) The first sewer

rental law. It did not contemplate the issuance of sewer bonds backed by the credit of the municipality and to be paid by millage levy; rather the sewer rentals could be substituted for all or any part of the indebtedness. This made a very flexible provision. However, this particular sewer rental law did not meet favor by those cities who were interested in financing sewage works but had reached or approached the limits of indebtedness. It did not provide for any new type of security to be issued. It was simply a new method, and a more equitable method of retiring general obligation sewer bonds. (4) At the last session of the legislature emergency legislation was passed which permits the issuance of revenue bonds which are not backed by the full faith and taxing power of the municipality. These bonds are backed by a lien against the property itself. The utility itself is security for the bond, and rentals may be collected for the retirement of the bonds. This created a new type of security. No such bonds have yet been issued in the state of Iowa. I know of no attempt to sell them.

Those are the four means of financing open to city councils. A combination of them may be used, except the last which does not so provide. These four will cover almost any conceivable situation. Any city or town can, if it so desires, finance legally any reasonable project.

In discussing the subject of this paper, I should possibly show methods of breaking down the sewer rental law into particular cases. I don't know that there is any general ordinance that can be applied to sewer rental ordinances. Each city must consider its own needs and own conditions before it can lay down rules. However, the complications in our own town of Cedar Rapids offer as good an example as any for opening the discussion. We have an industrial town. Several industries have their own water supply either because it is cheaper or because they need soft water. The soft water supply now being produced by the city is being used more and more. The growth of our city water consumption will exceed the growth of private water developments. We are confronted with this combination. The domestic sewage from the water consumed amounts to around 6,000,000 gallons per day. We have several industries pumping from 100,000 to 15,000,000 gallons per day for their own purposes. A good deal of that privately produced water is used for condensing purposes and does not add to the treated waste flow. About 30 per cent of the sewage flow in Cedar Rapids is from privately produced water put through industries. The 70 per cent is the used water supply of the municipal system.

The first question we had to decide was whether the industries should be included with the city sewage at all. We decided that the entire community should be treated as a unit. The cost will be distributed between individuals and industries. We first figured on the probable cost of a

sewage treatment plant to handle the city waste, exclusive of the major industries. Then the difference between this cost and the cost for sewage treatment for the whole community was charged to the industries. Obviously we might reverse that method of estimating and figure on the cost of a plant for industries and count the additional cost for the city. This would probably almost reverse the amount charged to industries. But we feel that it is legitimate to encourage local industries, just as far as it causes no additional burden to the property owners within the city. The estimated cost for the city is the amount the taxpayers or water users would have to pay anyway. On the basis of this estimated city sewage cost, we set up the schedule of rates for the city in general. We then charge to the major industries a rental which will pay for the additional cost of financing and operating the plant. We feel that we have played fair with both city and industries.

It is a peculiar thing that in figuring back from such a basis, the computed cost per million gallons of sewage treated is almost identical for industries as for the individual property owner.

The next step to break the rate down for individual users was to consider whether a straight percentage increase in water rental should be charged, or a flat unit price for thousands of gallons of water consumed. It is obvious that in the average rate set up in water supply industries there is a sliding scale. The cost of the first unit consumed is comparatively high, and the scale keeps descending as the amount goes up either per month or per quarter. However, this is different in regard to sewage treatment. The cost of treating the tenth million gallon of sewage does not decrease as does the production of the tenth million gallon of water. We figured the cost of operating the plant and retiring the bonds in 20 years, and found this could be done by adding 17 per cent to the water rentals. The large and small users both pay an additional 17 per cent of their water bill. The price for water consumed will be reduced as the amount increases. We first figure what the gross percentage increase in the water rates will be and then figure that on the bill. We may be able to retire our bonds at a more rapid rate and then we will be able to reduce the rental rates.

Discussion

Mr. Hopkins: Have you ever investigated the Fort Worth rental schedule? I understand that they were one of the first large cities to use this financing plan.

Mr. Green: Most of the Texas plants are on flat rates, which I do not think is particularly to be recommended. If your city water system is not all metered it is an imposition upon the non-users for the public has to pay the surplus bill.

Mr. Hopkins: Will you have your entire water supply metered or will you attempt to measure the water in some other way?

Mr. Green: We expect to make no measurements except in accounting work. The privately produced water will have to be gauged.

Question: How do you take care of the fact that many industries have sewage that is relatively difficult to treat?

Mr. Green: That is taken care of in the differential cost. We set up the scale for our major industries just in proportion to the additional cost which may be charged entirely to them, and charge the rentals on operating costs on a volume basis. These industries produce about 35 per cent of the entire sewage. We have some firms which would pay as high as $1500 a month, and some $100 or $200 a month for sewer rentals.

There is something that I would like to bring up that was mentioned in Mr. Hopkins' talk this morning. Suppose revenue bonds are issued on a sewer rental basis, is there any way that a city council can legally agree to maintain a schedule of rates which will be good security for that paper? I understood that no long term contract to maintain certain rates could be entered into by a city council or other rate making body. If that is true, I don't see how the government could approve of these bonds, or how any bond house would recommend such paper to their purchasers.

Mr. Stipp: If the city council agrees to maintain a rate which would retire the bond, that would make it a general obligation of the city.

Question: If you are up to your bond limit, and can't issue general obligation bonds, and the government will not accept revenue bonds, can you not use the leasing basis?

Mr. Hopkins: I think that the government is going to be rather loath to enter into the leasing proposition in most cases. There are some unusual cases where the government could do this very well. Suppose a city is located near the state line and the only desirable water supply for them is in another state. Now cities are creatures of the legislature and have only those powers which are delegated to them by the statutes. Obviously they could not go into another state and condemn land for their water works. The government, however, could do it. It could take this property and lease it to the city.

EXPERIMENTS IN TRICKLING FILTER VENTILATION

By Max Levine

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A trickling filter is a mass of rock or other material on which bacteria find an opportunity to grow and live and transform the sewage constitu-

ents which are brought to them; it is a factory for transforming constituents which are considered objectionable to materials which are not so objectionable. A filter, therefore, is a mass of rock properly and adequately aerated. If it is not so aerated the best results cannot be obtained. Unfortunately there has not been an adequately extensive series of studies with reference to the significance of filter ventilation.

Examination of a number of standard works on sewage treatment with a view to obtaining some idea as to what well-known engineers think about filter ventilation was not very illuminating as to whether or not a draft of air is needed or, if there is a draft, whether it enters from the top or bottom of a filter or from both apertures. The authorities do not agree and the reason in part, at least, is that we are still in a state of opinion, and lack information based on carefully controlled experiments. That air is necessary for oxidation in filters is a well known fact, but the question of how much air is necessary and the sources of this air in a filter are in need of considerable clarification. The manner in which air finds access into the interstices of a filter (that is whether it enters from the top or the bottom or both ends) is a matter of some theoretical and practical importance. The cost of construction, for example, would be influenced in part by an answer to this question. If we do not need ventilation from the bottom, then possibly extensive systems of underdrains may be dispensed with and the cost of construction thereby reduced. If ventilation from the bottom is indispensable, then information on the extent of such ventilation should be of interest and value. Several views have been expressed in the literature but considerable question and ambiguity still exist.

Metcalf and Eddy are inclined to the view that downward ventilation, incidental to the natural flow of sewage through the beds, is adequate, if liberal provision is made for free drainage. Adequate drainage, however, would probably involve ventilation from the bottom.

Fuller and McClintock express a similar view, though somewhat qualified when they say that oxygen must be constantly present in all places in a filter, and that maintenance of such conditions depends mainly upon movement of the air down through the pores of the filter. In view of the qualifying word, "mainly" the question as to the source of "the minor" fraction of the necessary oxygen is left open.

Hatfield reports that with complete submersion of the underdrains for a period of two to three weeks, the effluent did not deteriorate seriously, and Buswell, finding that the amount of free oxygen in different depths of the filter was not markedly different, is inclined to consider that diffusion alone is sufficient to supply the necessary concentration of oxygen in filters.

Quite in contrast to the foregoing views, that surface ventilation is

sufficient, is that of Phelps, who states that filters are preferably built above the ground with as much ventilation through the side walls as possible, and recommends ample provisions for ventilation.

Thus there are expressed the views that all needed ventilation may be obtained from the top alone, that additional ventilation through the sides is desirable, and that bottom drainage, at least (and consequently bottom ventilation), is essential.

In the course of some observations on experimental trickling filters a rather simple device was constructed to see if enough air could be obtained from the top or bottom if ventilation was also desirable. The filter consisted of a metal box two feet square and six feet deep with the bottom consisting of a grating of bar screen. To this bottom was attached a hopper which was provided with two vents on the sloping sides and which terminated in a water sealed trap so that no air could get in from the bottom. The sides then were closed. If the vents were open there was ventilation from both top and bottom but with the vents closed air could enter only from the top.

The filling material consisted of coarse cinders 5/8 inch to 2 inches, and the filter was dosed with a milk waste at a rate of 840,000 gallons per acre per day for 14 to 16 hours, and rested from 8 to 10 hours daily. This cycle would favor maximum diffusion of air during the rest period and simulates conditions which are encountered in practice in small dairies. The applied waste consisted of 1 per cent skim milk (B.O.D. from 450 to 500 p.p.m.) until the filter had ripened; after which the concentration was doubled, and the vents in the hopper closed. Operation under the latter condition (bottom vents closed) for about four weeks was followed by a similar period during which the bottom vents were opened, and thereafter for periods of 204 weeks the bottom vents were alternately closed and opened.

A consideration of the results will be restricted for the present to the determination of nitrates, reaction of the effluent and oxygen demand.

The ripened filter with the bottom vent open and receiving 1 per cent skim milk produced an effluent containing 12 p.p.m. of nitrates. On closing the vents and at the same time doubling the concentration of skim milk, the concentrations of nitrates in the effluent dropped to 0.8 p.p.m. Again opening the bottom vents caused the nitrate content of the effluent to rise to 25 p.p.m. and this was followed by a drop to 2.4 p.p.m. when the vents were again closed. Alternately opening and closing the vents thereafter gave effluents containing 12.8 p.p.m. (open), 3.82 p.p.m. (closed), and 5.64 p.p.m. (open) of nitrates respectively. It was evident therefore that bottom ventilation was beneficial to nitrification and that the recuperative powers of the filter were progressively deteriorating as time went on.

During the periods when the bottom vents were open the effluents were alkaline but on closing the bottom the effluents became progressively more acid.

The effect of bottom ventilation on B.O.D. of the effluents was particularly noticeable. A B.O.D. of less than 15 p.p.m. rose to 150 p.p.m. during the closed period, fell to less than 10 during the second open period, rose again to 480 on closing the bottom vent, again dropped to about 10 on opening the bottom vent, only to rise to 450 p.p.m. a few days after bottom ventilation was shut off.

As far as these experiments are concerned it must be evident that bottom ventilation was very beneficial. Whether the results with a filter 2 feet square are applicable to full size installations is of course problematical, but in this connection it is interesting to note that in some experiments with packinghouse wastes the results obtained in experimental filters 3 feet square were practically duplicated on a half-acre installation. On this filter it was observed on two occasions that interference with bottom ventilation (in one instance obstruction of the vents by snow and another instance clogged screens over the filter galleries) interfered with filter purification efficiency.

In an experimental filter which was artificially ventilated through the bottom and which was dosed with packinghouse wastes at 10 million gallons per acre per day, it was found that cessation of forced bottom ventilation resulted in very rapid clogging. In view of the results obtained it is felt that the bottom should be constructed so as to permit abundant opportunity for natural bottom ventilation.

Under the conditions of operation of the experimental filter for creamery wastes here reported, the rest period of 8 to 10 hours permitted abundant opportunity for diffusion of air to take place when the filters were being operated with the bottom vents closed. The evidence is therefore quite convincing that bottom ventilation was very beneficial. Attention should be called to the fact that the filling material was finer than is generally employed, and that the concentration of waste was higher than what is encountered in domestic sewage, but the results nevertheless demonstrate the necessity and desirability of adequate bottom ventilation.

SEWAGE SEDIMENTATION

By A. H. Wieters

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In spite of the fact that sedimentation was the first process used for the purification of sewage, perhaps less is known about it today than any

other step in the process. In the first place in a sewage tank, whether it is Cameron, septic, Imhoff, or one of the later types there is only one thing attempted:—to settle out as much suspended material as is possible. Solids are divided into three classes, (1) suspended, (2) colloidal, and (3) dissolved. There is no strict line of demarcation between these different classes of solids. We can't, basing our information on sizes of particles, say just when a solid ceases to be a suspended and becomes a colloidal solid, nor when a colloidal becomes a dissolved solid.

A great many years ago a man by the name of Stokes evolved the theory of sedimentation and formulated it mathematically. About thirty years ago Allen Hazen applied the Stokes theory of sedimentation to sewage and formulated a mathematical theory as applied to sewage. There are so many factors in the application of these theories that many of us have lost sight of the fundamental features. We are inclined to base our designs entirely on rule-of-thumb methods based on experience but we should not lose sight of the general theory of sedimentation, and thus avoid some of the ridiculous mistakes that are made. I am not going to attempt to go into the mathematics of sedimentation, but shall give you the Stokes theory in a general way, discussing the basic factors upon which it depends. There are limitations which prevent this theory from working out practically, and that is particularly true in dealing with sewage when you have solids in so many different forms.

In formulating his theory of sedimentation, Stokes based his work on silt particles. These are rounded particles whose specific gravity is known. Most sewage solids do not come in this class. In consideration of his problem he tried to find just what it was which caused these particles to settle and what factors were working against them. It is the pull of gravity that causes a particle to settle in a tank. Operating against the gravity are other factors, depending on the size of the solid. Some factors are more important than others. Stokes was dealing with small particles, $\frac{1}{10}$ of a micron in diameter. In dealing with such small particles the viscosity of the water is an important factor. In sewage it is not so important. We all know that molasses or lubricating oil will flow more quickly in summer than in winter. In cold lubricating oil a particle of sand will not settle as rapidly as it will in warm oil. We know that viscosity varies directly with the temperature. In dealing with large particles in water the effect of viscosity is small and can be ignored. But with the smaller particles viscosity becomes an important factor and is directly proportional to the temperature.

Another important factor is the specific gravity of the particle as compared to the specific gravity of water. Sand has a specific gravity of 2.65, as compared to 1, the specific gravity of water. A sand particle, therefore, would settle rapidly. In sewage we have all kinds of specific

gravities, from particles of quartzite to particles with a specific gravity less than that of water. In Hazen's work on sewage he took as the ordinary specific gravity of sewage particles 1.2, or slightly more than that of water. Consequently those particles will settle slowly. We have to consider, therefore, the specific gravity of the particle to be settled, the specific gravity of the water (hard water does not have the same specific gravity as pure distilled water), the temperature, and the diameter of the particle. The smaller the diameter of the particle, the less area will be exposed to the water; in the larger particles the greater will be the area exposed to the water as compared to the weight, and the greater the friction between the water and the particle itself, and the less readily it will settle. Stokes stated a formula in which he expressed, mathematically, the speed with which any particles are settled as a function of gravity, temperature, specific gravity of water and diameter of particle. Hazen's formula applied to a sewage particle about 1 millimeter in diameter. The effect of the viscosity of the liquid must be dropped out of a formula dealing with particles of this size.

Hazen also proved mathematically that settling in any basin is dependent upon the area or surface. The velocity with which a settling basin is operated is important and also whether the basin is operated continuously or intermittently. Liquid should not be introduced at such a rate that it will pick up particles which have already settled. In a certain time the particles will settle a certain distance if the velocity is below that necessary to lift them again. The larger and heavier particles will have settled more rapidly than the smaller particles with a lighter specific gravity. These will be distributed through the liquid and at the outlet of the settling basin you will find the clearest, cleanest water in the top half inch. Only those particles which do not readily settle in the time taken for the water to come across the basin and those with a very light specific gravity will be left. Therefore if you draw off a thin film of water you will get the water most free from suspended solids. A few years ago a great many engineers had the idea, and some still do, that when new liquid was introduced the ideal situation was to displace the liquid. The theory you hold makes a difference in the design of the tank. It will also make a difference whether you consider surface area important. In a ten-foot tank you would have twice the detention period you would have in the same area in a tank only five feet deep. You would get exactly the same clarification in a five-foot tank as in a ten-foot tank, for it is a function of area rather than depth. In a five-foot tank, however, you will get wind currents and other factors which operate against the Hazen theory. Temperature currents are also something that we know very little about. We know that warm water will rise to the surface. A body of liquid is usually colder than the warm

sewage and you must battle that in some way. If you do not the introduced warm sewage will flow across the top and not settle down into the cold fluid. In the summer, the surface of an open tank is likely to be warmer than the incoming sewage and then the sewage will flow toward the bottom. Those are questions that have bothered engineers ever since the beginning of sewage treatment. The currents may be controlled by various kinds of inlets and outlets and baffles, but you can build a certain type of sewage tank in one place and it will work fine, and under apparently the same conditions build a similar plant and get poor sedimentation.

There are many differences in sewages which operate against the proper functioning of the tank. It is important that you get uniform distribution of incoming sewage and uniform distribution at the outlet end. You should have a weir as long as possible. The longer the weir the thinner the film you can shear off, and you are not increasing the velocity carrying out materials that have already settled. The theory of a lower weir is more applicable to muddy water where you have particles of silt and quartzite. In sewage there is generally scum on top, for there are some light particles that will float, and there should be some skimming before the water goes out on the filters. That scum baffle should be made as shallow as possible. If you build it too deep you are partially defeating the purpose of the weir.

The tendency is now toward the building of shallow tanks. The fact is recognized that regardless of the theory of the displacement of liquid, it is practically impossible to get satisfactory displacement in a sewage tank. The tendency is towards tanks of larger area and shallower depth. It is cheaper to build a ten-foot tank than a five-foot tank of the same volume. It has been our observation that there is very little if any benefit by building deeper than to eight or nine feet.

There have been various devices to get an equal distribution across the tank. One of the common devices is simply to introduce the sewage by means of an overflow weir. Theoretically that ought to give you almost ideal distribution; practically, it has a number of limitations. The incoming sewage is usually warmer than the liquid in the tank. If the sewage is not deflected down into the liquid it will flow across the top. In order to break up that velocity a baffle is inserted which causes a change in the direction and breaks up the tendency for short-circuiting. Another device is to use a trough with holes in the bottom. That has about the same effect as an overflow and baffle. Another device which I think is very good is a trough with an "L" inserted below the inlet which drops the sewage back against the wall and makes a complete change in the velocity. I don't know the proper depth at which sewage should be introduced. There are several opinions on that. At the out-

let end, I think you can't beat an overflow weir, and I believe that the longer weir you can get the better the settling. Some of the latest settling tanks are circular in design, with the sewage introduced from the center and the overflow weir all around the tank. Theoretically, that is the best type that has been introduced so far, by virtue of the fact that you have the maximum weir that you can have in a tank. In a square or rectangular tank the settling may be vastly improved by extending the weir on three sides.

In the Cameron tank the sewage comes in at one end and goes out the other with the sludge settling to the bottom. This gave some difficulty because the sludge in digesting gives off gas, and the gas interferes with the settling. The Imhoff tank was next designed to separate the settling from the digesting compartment. The arrangement was simple. The solids, once settled, slid down to the bottom of the tank, and the gas was deflected by an overlapping wall so it could not interfere with settling. There are many Imhoff tanks that are giving satisfactory service today. The next development was getting back to the plain tank, circular or square, with some mechanical device to remove the sludge before the material started giving off gas. That type of tank is probably used more than any other at the present time. The difficulty is that it requires operating care. One of the latest developments, depending on the principle of surface area, is a tank with trays. A tank with 5 trays has an arrangement so that ⅕ of the flow goes into each tray. You have, in effect, five times the area you would have without the trays. There is a mechanical device for removing the sludge with a pump. I have not seen a tank of that type and I have heard favorable and unfavorable reports concerning it. The makers claim that these shallow trays have the effect of coagulating the finer particles and settling them out.

There have been many attempts made to coagulate the finer particles in sewage. One of the oldest devices, which did not give a very satisfactory effluent, was common salt. With the best settling, we get about 75 per cent of the suspended solids. We are doing very well indeed to get that percentage. The suspended solids are the only ones which will settle. One of the first types of treatment was to use some chemical to coagulate the other solids and get better clarification. But ordinary domestic sewage has about 50 per cent of organic material in dissolved solids, so the effluent is still not satisfactory after the suspended solids are removed. It was usually found that so much chemical was required that the process was very costly. Providence, Rhode Island, was one of the first cities to use chemical precipitation. The method seems to be coming back into favor again. There have been some experiments, notably in New Jersey, which have shown that the use of much smaller amounts of chemicals may be used with better results.

One of the chemicals used for clarification is aluminum hydroxide. But there the very nature of the material suggests that the finer particles of matter are removed by mechanical action, and that it is a sweeping out process. With the experiments that are now being made in regard to chemical clarification, I look for many developments along that line.

BIOLOGIC FILTRATION

By W. E. Galligan

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Sewage farming and sub-surface irrigation are antecedents of biologic filtration. The difficulties experienced in the operation and maintenance of these older methods of sewage disposal have led to the development of the sewage filters as we know them today.

Land irrigation by sewage was associated with the earliest work in sewage disposal and it was natural that this should be so. The average amount of sewage that can be placed on land and disposed of in connection with the growing of crops is 8,000 gallons per acre per day, a very small amount as compared to the rate of application on either an intermittent sand filter or a trickling filter. If this method were used in disposing of the Ames city sewage, about 100 acres would be required. Compare this 100 acres with the three-quarters of an acre now used for this purpose.

It is true that there is potentially available a considerable amount of fertilizer in the sewage from a city the size of Ames but it is a mistaken idea that very much of this is available as plant food. Before nitrogen can be used by growing crops it must be in the nitrate form and since a large amount of bacterial work is required to oxidize the nitrogen in sewage to this form, complications soon arise when an effort is made to raise crops and dispose of sewage at the same time. The history of sewage farming shows that the main objective always was sooner or later the proper treatment of the sewage rather than the production of crops. Sewage sick farms were the rule rather than the exception and this illness inspired a corrective treatment resulting as was said before in our present day sewage filters. I do not intend to make this a paper on sewage farming but it is necessary to understand the difficulties encountered in this practice in order to get a clear picture of the limitations and capabilities of intermittent sand and trickling filters.

The development of filtration of sewage was along the most natural lines. Suppose that instead of having your sewage filter, whether sand or gravel, you had a large area of land on which the sewage from your

Imhoff or septic tank had to flow. Sooner or later the surface of the ground would become clogged and in digging down into the soil you observed that it was sour and ill smelling, what conclusions would you reach? You perhaps would decide that if this ground could be plowed up so as to allow air to penetrate into the deeper layers it would dispose of more sewage. You would also see that grease and soap, as well as other solids, were collecting on top of the soil and around the particles of earth and a surface clogging was taking place. As a matter of fact these two observations would lead you into the very same steps that led to the development of first the sand filter and next the trickling filter.

Biologic filters are of three types, intermittent sand filters, contact beds and trickling filters. We might even include activated sludge in this category since it has been rather well said that the activated sludge process consists in passing a filter through a liquid rather than passing a liquid through a filter. This would be stretching the point, however, as far as this paper is concerned; therefore we will consider only the first three mentioned.

General Features of Sewage Filters.—The authors of textbooks in describing the general features of sand filters usually associate the intermittent sand filter with areas or districts in which sandy soil and natural sand deposits exist. This was strictly the case when the first biologic filters were built in the United States because it happened that where the greatest need for sewage treatment prevailed, there was as a rule sandy soil.

We do not usually think of Iowa as being a sandy soil state nor are there extensive sand deposits which might be cleared of loam overburden and used as an intermittent sand filter. Nevertheless there are many sand filters treating sewage in Iowa and they are doing the job in a manner that should make the best coarse grained filter ashamed of itself if the standard of achievement were a highly clarified and purified effluent. Dean Anson Marston of Iowa State College built the first sewage treatment plant in the State of Iowa which was the sand filter type, and followed this with others, several of which are working satisfactorily to this date.

In taking the step in biologic filtration from the discharging of sewage on land to the treatment on sand filters let us understand what difficulties were overcome. First, more attention was given to the character of the beds as regards the pore space in the bed and hence the greater the ability of the filter to breathe or acquire an oxygen condition throughout the bed. This was a result of the realization that the ordinary soil soured too quickly in the absence of air. A uniform size sand was recognized as being the one that furnished a very large pore space

and one that worked well even when very much larger quantities of sewage were applied to the bed. The second difficulty experienced in sewage farming was overcome by removing the dirty top layer of sand whenever it became clogged. Hence we have the transition from the first attempt at biologic filtration to the use of sand filters. Better methods of removing the treated sewage were devised as time went on and means of applying sewage intermittently were developed, until the sand filter assumed the form that I will describe.

The filtration area is usually divided into a number of rectangular beds. Successive beds or rows of beds are divided by embankments. Each bed has a trough or pipe running along its long axis through which is discharged at equal intervals along the length, uniform quantities of sewage. This sewage is discharged intermittently upon the sand surface, filters downward through the sand and is collected in underdrains. The receding sewage draws air into the sand and thus maintains aerobic conditions in the filter. This type of filter is foolproof in its operation even though raw sewage is applied without clarification since the solids in collecting on the top of the filter form a mat and reduce the capacity of the filter until the dirty mat is removed. If clarified sewage is applied to the bed the penetration of the smaller particles reaches a greater depth and in so forming a mat, more sand must be removed when the filter has to be cleaned. Possibly the greatest danger in operating a sand filter can result from the practice of stirring the sand when it becomes clogged, because this stirring, plowing or raking of the sand will allow the clogging particles to penetrate the interior of the bed where eventually the sand will assume the same characteristics as the land on which sewage farming was a failure.

The nominal rate of treating sewage on sand filters is not greater than 150,000 gallons per acre per day; therefore in line with the characteristic American desire to step up the rates of everything, a new type of filter was sought so that smaller areas could be allocated to sewage treatment plants. Of course this was particularly true in localities in the United States where land areas for enlarging sand filters to care for increasing population were scarce and expensive to obtain.

This desire led up to the adoption of the first coarse grained filter, the contact bed. In this bed the rate of treatment was increased to about 600,000 gallons per acre per day as compared to the 150,000 gallon rate. Unfortunately however in attaining this increased rate some sacrifices had to be made in quality of treatment. Instead of the effluent being crystalline clear and very highly purified, as in the case of the sand filter, it now became slightly turbid with material discharging from the bed and its degree of treatment was appreciably reduced.

The contact bed consisted of a water-tight tank filled with coarse

grained crushed rock or gravel similar in size to that used in trickling filters today. This tank was usually about five feet in depth and constructed with pipes for leading the sewage in at the top, and for discharging the sewage from the bottom. The tank was filled with sewage, allowed to remain filled for two to four hours and then emptied. A rest period of perhaps six hours was provided and the process repeated. The filter medium in this filter gradually became clogged and in time the contents of the tank had to be removed, hand washed and replaced. This operation probably was repeated on the average once in three years.

Still not satisfied with the rate of treatment, more experimental work was done and the trickling or percolating filter made its appearance. In this type of filter better means are provided for aeration and distribution of the sewage on the bed but again at the expense of not such good results from the standpoint of treatment. This filter handles about six times as much sewage in a day as did the contact bed and it handles about twelve times the volume as did the sand filter for the same area. The effluent is considerably inferior to that of the sand filter, for two reasons. At times when this filter unloads or sloughs the effluent carries large quantities of the humus-like material that results from the biologic life that exists in the filter. While this solid material is largely harmless it is the source of considerable complaints downstream from the sewage treatment plant and therefore must in many cases be removed before the plant discharge is accepted as suitable. The other reason is the fact that a larger body of dilution water is required to dispose of the same amount of effluent from the treatment plant in a suitable manner than is required if the sewage is treated in a sand filter plant.

The trickling filter is made up of a bed of coarse stone or gravel that has a size of between one and three inches. The beds range in depth from five to ten feet, the greater depths being used where it is desired to accomplish a higher degree of treatment. The sewage is applied to the bed at as uniform a rate as is possible, at intervals of from five to fifteen minutes. An elaborate system of underdrainage is provided as compared to the underdrains for sand filters. Concrete bottoms are built in this type of filter whereas the underdrainage pipes were laid on the ground in the case of sand filters. Flies and odors are by-products of trickling filters while their more graceful contemporary, the snipe, inhabits areas used for sand filters. The one big advantage in the use of trickling filters is that they automatically clean themselves if they are well designed and carefully operated, whereas in the case of the sand filter it is designed to clog and must be cleaned at usually regular intervals. The trickling filter demands a clarified influent while the sand filter will work without injury to itself if the sewage receives no pretreatment whatever. I do not mean that preliminary treatment is unde-

sirable but the point I want to make is that unless the sewage applied to a trickling is relatively free from larger solids the filter may be seriously damaged and this is not the case with the sand filter.

SLUDGE DIGESTION

By C. H. Currie

Currie Engineering Co., Webster City, Iowa

Until comparatively recent years, but little research or study had been given to sludge digestion. The operator's troubles possibly had more to do in awakening the interest and thought of the designing engineer than did any other factor. Today, almost every publication relative to sewage treatment contains one or more articles on the developments and findings in sludge digestion studies.

What is meant by sludge digestion? Fresh organic sewage solids, decomposing under water without proper seeding material, go through a process of acid fermentation or putrefaction. The sludge is yellowish green or greenish in color and is sticky. It is characterized by the violent odors produced. Entirely different from this acid fermentation or putrefaction is the second kind of fermentation, which soon establishes itself in properly controlled digestion tanks. The process is almost odorless. This alkaline or methane fermentation is the digestive process sought in sewage works.

In starting new sewage plants, unless proper bacteria can be introduced into the fresh sludge, a considerable period of time will elapse before proper sludge digestion takes place.

The first attempts to better sludge digestion were directed toward increase of digestion capacity or volume and use of chemical control. In the septic tank, little can be done to improve digestion, except in capacity. It would be impractical to collect gases of digestion from the municipal septic tank and burn them to destroy the odors. It is somewhat difficult to exercise artificial control of sludge digestion in an Imhoff tank, although the gases can and have been collected and utilized. The more recent developments from sludge digestion studies have resulted in a trend toward the collection of sludge in one mechanically cleaned tank and digestion in a separate compartment, where heat may be used as a means of improving and intensifying the biological processes. Extensive experiments on heating the sludge to various temperatures have shown increased possibilities in hastening digestion.

Active intermixing of raw sludge with digesting sludge under temperature control has opened another valuable chapter in the sludge digestion field. Manufacturers have developed equipment better able to

handle sewage gases than was at first obtainable, such as meters, scrubbers, coolers, and other collecting and utilizing devices.

The routine operator's procedure in sampling and analyzing sludge, to determine what is necessary to improve conditions, has been greatly simplified and clarified. Operators often find that sludges bulk and the overflow liquor contains a considerable amount of sludge. Sometimes speeding up the stirring mechanism has eliminated this problem. Acid sludge is neutralized with lime.

One of the troublesome problems in separate sludge digestion is the handling of the supernatant overflow liquor. This has become so acute that in a few plants this overflow liquid has had to be handled separately from the ordinary liquid-treating methods at hand.

My picture of what separate sludge digestion may comprise in the future would consist of a comparatively small heated digestion tank, large enough to handle sludge of a community or industry for from 15 to 25 days, with mixing devices for rather violently mixing the raw incoming sludge with the digesting sludge, permitting the displaced liquid and sludge to flow to another and larger sludge digester, which could or need not be provided with gas collection facilities nor with heating facilities and permitting this sludge, intimately impregnated with digesting bacteria, to carry on the final stages of digestion.

The supernatant liquid from such a combination of digesters should be much improved and more easily handled with the incoming fresh sewage.

With the average sludge in a heated and agitated sludge digester, experiments indicate that 80 to 95 per cent of the gases developed through digestion will be produced during the first 21 days or less. Another possibility in connection with this suggested layout would be that the heating coils would not be placed within the digesting tank.

If the intermixing were accomplished by pumpage, withdrawing sludge and liquid and returning to another part of the tank, to accomplish the mixing desired, the heating coils could be outside the tank and accessible for the removal of the adherent insulating sludge.

More knowledge and constantly better sludge digestion may be expected as the result of the world-wide thought and study that is being given this subject at present, not only from improved mechanical control, but from possible future developments in better chemical control.

Sewage plant operators throughout the country are entitled to a considerable measure of credit in the awakened interest and study that has been given this phase of sewage treatment.

GENERAL OBSERVATIONS COVERING THE AURORA SANITARY DISTRICT PLANT

By Walter A. Sperry

Supt., Aurora Sanitary District, Aurora, Illinois

The Aurora plant was among the first to make use of the separate sludge digestion plan with gas collection; it was placed in operation September, 1929, designed to serve 67,000; it has a present connected load of 47,000 persons. The equipment consists of an automatic screen —½-bar spacing, a detritor, 4 traction type clarifiers—50 feet square and 10 feet deep, 3 digesters of the bridge truss type—50 feet square and 16½ feet deep and 10 under-drained sludge drying beds—40 by 125 feet. The trickling filters approximate 4 acres with six feet of freeze-tested limestone and bottoms. Four sets of syphons arranged in two dosing chambers serve the four filter sections. No gallery system and no secondary treatment are provided.

The plant is located some four miles out of Aurora, with sufficient fall for primary treatment by gravity. The pumping equipment, therefore, is installed at the plant and between the clarifiers and the filters. Since the flow in Fox River averages 800 c.f.s. providing an average annual dilution ratio of 1 to 70 and a winter ratio of 1 to 90, advantage is taken of this fact to eliminate the filter operation for about 6 months in the year. This saves about $2,500 of operating expense for power annually.

All the structures are housed in tile roofed, ivy covered buildings of Spanish type. The grounds, comprising some 14 acres, with many of the original trees, have been landscaped with native shrubbery and flower beds, the unused portion being treated for bird reserve purposes.

The intercepter system includes 10.9 miles of pipe ranging from 6 to 69 inches, the plant receiving combined sewage, relieved of much of the storm water uptown at separatory type manholes.

To date the completed intercepter system cost $751,800 and the plant proper $675,000 making a total investment of $1,426,800.

The following table summarize the main characteristics of the raw and treated liquors for 1932:

Table A

Average flow	7.2 M.G.D.
Screenings	1.3 Cu. Ft. per M.G.
Grit	1.87 " " " "
Grease and scum	0.42 " " " "
Equivalent dry solids to digesters per day	4,000 pounds.
Percent dry solids in sludge	5.6%
Pounds oxygen available in Fox River	48,000 pounds per day.
Pounds oxygen required by the effluent	2,830 " " "
Average dilution ratio	1 : 71
Cubic feet gas per day	33,800
Average B.T.U. value	630
Cubic feet gas per capita per day	0.723

TABLE B—*Parts Per Million*

	Raw	Clarified	% Red	Filters	% Red
Settl. Solids	2.2 c.c.	0.5 c.c.	77.2	0.2 c.c.	90.8
Susp. Solids	140	60	57.2	26	81.4
5 Day B.O.D.	102	74	27.4	25	75.5
Oxygen Consumed	58	48	17.3	34	41.4

Pounds oxygen per capita in raw sewage 0.130
Pounds suspended solids per capita in raw sewage 0.178

TABLE C—*Costs for Operation Only for 1932*

Payrolls	$16,964.27
Power	5,370.79
Truck	95.01
Phone	126.52
Office	198.54
Laboratory	340.44
New equipment	775.29
Repairs	283.25
Miscellaneous	1,211.20
Total	$25,865.31
Cost per Million Gallons	$9.76
Cost per Capita per Year	$0.55

Notes on Special Features

Incinerator.—In 1931 an incinerator was installed in the screen house. This should be basic equipment in every plant. It is gas-fired at no cost and consumes without odor all screenings, grease and scum. Planned to burn 8 cubic feet of material, at received moisture content, in 4 hours on a gas consumption of 7,000 cubic feet per day it exceeded the guarantee by 25 per cent. Not only have our burial problems been eliminated but there is neither sight nor smell of screenings to shock the sensitive visitor.

Storm Overflow.—Prior to the fall of 1932 storm water was discharged from the weir in front of the automatic screen. Since then the original storm weir has been raised and a new overflow weir built to discharge storm surplus after screening and grit removal. In thus further protecting the receiving stream we have improved our public relations as well as our operation and are in line with advanced practice.

Sludge Meter.—As originally planned there was no way of metering sludge to the digesters except by time and an assumed pump capacity. We have completed the design and construction of a substantial sludge meter of the "tipping bucket" type including an automatic sampler. This gives a dependable check on both the volume and quality of sludge

to the digesters. Home made, it cost about $100. To date we have found that we were in error using the former method as much as 18 per cent.

Screens to Protect Filters.—Recently we followed the practice at Schenectady, New York, and Bloomington, Illinois, by installing a ½ in. mesh screen in the dosing chambers to protect the filter nozzles from miscellaneous clogging material. While these require daily attention and become annoyingly coated with a surprising amount of hair, which makes them difficult to clean, we expect them to be worth their keep in reducing the labor of nozzle cleaning.

Settling Tank Baffles.—The outstanding improvement in plant operation during 1932 was the introduction of baffles on the inlet side of the clarifiers. We were suffering from suspended solids removals averaging but about 40 per cent. Laboratory and tank studies indicated the difficulty to be the familiar one of unequal distribution of the raw sewage both to and through our tanks. Weir and gate adjustments corrected the former and the above baffles corrected the latter. The inlet channel, 50 feet in length, was fed from one end. The input velocity, however, was great enough to set up a cross current beneath the inlet channel strong enough to throw the bulk of the tank discharge over but a part of the discharge weir, leaving one-third to one-half of the tank "dead." The introduction of baffles under the inlet channel on about five-foot spacing intercepted this current and directed it more uniformly across the tanks, raising their effectiveness from about 40 per cent to 57 per cent. Sludge volume, gas volume and quality of effluent to the river all reflected the wisdom of this improvement. The over-all effectiveness of the plant was raised by about 40 per cent.

River Control.—One of the "laboratory dividends" which make it possible for this plant to take advantage of our combination of gravity flow through primary treatment and the volume available in its receiving stream is the practice of recording the river flow in c.f.s. and the dissolved oxygen content as a laboratory routine. This enables us to report, every other day, the pounds of oxygen available in the receiving stream against the pounds of oxygen required by the effluent, as computed from the 5-day B.O.D. This expresses the fundamental bookkeeping between ourselves and our receiving stream and indicates when we may safely cease filter operation in the fall and when we should begin filter operation in the spring. This has successfully operated for three years. We have been interested in the fact that whereas a 30-day period might need to be anticipated for the filters to function properly, we have found this preliminary period unnecessary. For the past three years the filters have delivered a satisfactorily reduced effluent over the clarified liquor within the first two or three days.

Digester Invoices.—Once each month we take the trouble to sample all digesters at one-foot intervals, running the total solids, specific gravity, volatile matter and pH value on each sample. This enables us to keep a complete record of digester operation, thus we determine the loss in total solids and volatile matter, the content and general condition of all digesters. While it involves a very considerable amount of work, at the beginning of each month, we have found the record obtained a fascinating one. The results of such "invoices" in 1931, when compared with the averages of 1932 showed surprisingly close agreement. The 1932 invoice is shown below:

Total solids reduced	49.2%
Volatile matter reduced	67.0%
Average per cent solids in raw sludge	5.2%
Average per cent volatile matter in raw sludge	69.0%
Average solids above bottom layer in digester	9.2%
Average volatile matter in digester	45.0%
Cu. ft. gas per lb. of vol. matter added	12.6
Cu. ft. gas per lb. of vol. matter destroyed	20.0

Odor Problem.—Sometime during 1930 a joint suit was filed against the District for alleged personal and property damage due to odor totalling $50,000. Although the plant is located some four miles out of Aurora, there is a group of six summer cottages in a grove of trees on one boundary line about 1,000 feet north of the screen house. Another small group of residences is located on the main highway, lying west and south of the filters, and at an elevation of 50 to 60 feet above the filter bed level. Individuals from these two groups were joined by others from some isolated farm houses east of the plant, across Fox River at a considerable elevation and a half mile or more distant. Unquestionably these suits were not justified by the actual conditions prevailing at the time. They were instigated by two near neighbors and joined in with others whose location and amount of nuisance endured made the whole venture solely a financial gamble.

The source of such intermittent odor as existed came first from the unfortunate practice in 1930 of permitting raw gas to escape from the roof of the main building. This gas averaged 443 parts per million (23.3 grains per 100 cubic feet) of hydrogen sulphide. The second source was due to the release of dissolved hydrogen sulphide from the clarified liquor at the spray nozzles of the filters.

Roughly speaking the plant lies in a semi-circular bowl, whose edges are broken in two points—one north and one south from the plant—on the main highway along the west side of the grounds. The road on the east side of the plant is across the river and at the filter level. The odor

was never continuous, and appeared here or there depending upon the direction of the wind, except for rare occasions when the barometer was low and the air damp or still. Only occasionally was the odor more than mildly suggestive of hydrogen sulphide.

Such super-exaggerated tales as being awakened from sleep in the middle of the night and caused to vomit, the necessity of closing car windows and driving past the plant at 60 miles per hour and having the odor cling to one's clothing for two hours after driving past the plant would have been seriously offered in testimony had the case come to trial. Such is the peculiar psychology of the layman mind with respect to human wastes. Yet a nearby enameling plant and the Burlington sheep yards, capable of housing 75,000 sheep, for years have made the air, at times, heavy with a pronounced odor covering a large and populous territory, with only grumbling complaint.

Late in 1930 our Board of Trustees seriously considered the use of chlorine. This would unquestionably have remedied the difficulty satisfactorily. On the other hand there would have been a sizeable sum required for the necessary equipment plus an annual charge for materials, to say nothing of an additional operating hazard, the problem of shipping and receiving cylinders and the added labor for its proper control. Moreover, the situation did not seem sufficiently serious to warrant expenses estimated to be from $3,000 to $6,000 per year, until some changes in the existing operating methods had been tried out.

For the third year now we have operated without recourse to chlorine. Plant odors have been so controlled that no effort has been made to bring the suit to trial. The attorney in whose office they are filed remarked the other day, "they are getting a bit dusty." Each added year of present operation makes such a suit the more unlikely.

Our success in avoiding both legal entanglements and added operating costs lay in the following carefully planned schedule of plant operation.

First.—All unused raw gas has been burned before release.

Second.—The discharge end of the main intercepter, which lies near the summer cottages, has been closed with a light sheet-iron gate sealing at the water level and such gas as may have formed in the intercepter and which could be detected for some distance about the mouth of the intercepter, especially during the evening hours, is confined thereby and led through a 6 in. tile to the receiving box of the incinerator in the screen house and burned. This permanently deodorized the atmosphere in this vicinity.

Third.—In the original layout a 16 in. pipe and valve had been provided back of a dam in the city, through which river water could be drawn into the intercepter. During the summer period, when the filters are operating, from one to two million gallons of river water per day

are continuously drawn from the river into the intercepter. This tends to add some oxygen and hasten the flow so as to deliver a somewhat less septic sewage at the plant. The increase in pumping costs are much less than would be the cost of chlorine treatment.

Fourth.—The clarification period was cut in half by operating two of the four available clarifiers. This reduced the average retention time in the clarifiers from approximately 2.6 hours to 1.25 hours without seriously reducing the effectiveness of the clarification, and enabled us to deliver a somewhat fresher sewage to the filters.

Fifth.—The sludge blanket is kept well pumped down to avoid septic conditions developing as much as possible in the clarifier tanks—thus checking hydrogen sulphide formation.

Sixth.—Dissolved oxygen and hydrogen sulphide determinations as suggested by Salle and Reinke, using antimony tartrate, are made at 9 A.M., 1 P.M., and 4 P.M. as a matter of record and on the assumption that so long as the hydrogen sulphide concentration in the clarified liquor to the filters remains below one part per million no nuisance worthy of attention will develop. We give the results of these tests for August as generally typical, there having been but three-tenths of an inch of rainfall, so that the month was unrelieved by any dilution or flushing flow from the sewers. The flow for August averaged 6.4 million gallons, and the B.O.D. in the raw sewage averaged 116 parts per million.

Dissolved Oxygen and Hydrogen Sulphide Tests on 26 Days in August

Average results for the month in parts per million.

	9 A.M.		1 P.M.		4 P.M.	
	D.O.	H_2S	D.O.	H_2S	D.O.	H_2S
Raw	1.3	0	0	0.3	0	0.5
Clarified	3.1	0	0.8	Trace	0.6	0.3

At 4 P.M., hydrogen sulphide in the raw, 1 part per million or more on but 3 days.

At 4 P.M., hydrogen sulphide in the clarified, 1 part per million or more on but 1 day, and 0.5 part per million or less on 20 days.

Seventh.—In the meantime an extensive program of ground beautification in the form of flower beds, lawns, shrubbery and trees was provided which has turned the grounds into a local beauty spot. This investment has returned definite dividends with respect to odor control because neither enemies nor visitors can consistently *think* much of bad odor in the presence of the surrounding beauty. I sometimes think we have not attached sufficient importance or value to this last point.

This plan has not absolutely eliminated odor, but such odor as exists is only detectable during the evening hours of typical days when the barometer, wind, and humidity combine to produce typical odor conditions and then it is intermittent, short ranged and only mildly noticeable.

The Aurora Plan for Visitors

One of the most important duties of the responsible plant operator is to make his plant attractive, as well as instructive, to visitors. This statement is based upon the fundamental fact that the visitor is a "stockholder" whose dividend is the best results obtainable for the least outlay of *his* cash. Being a "stockholder" he has the right to know something about the "how" and "why" of *his* plant. Given this opportunity he becomes proud of his ownership and is ready both to pay for and defend the plant.

The following schedule details the Aurora plan for the reception and care of visitors:

A. A well planned road-sign names the plant and announces that visitors are welcome.

B. A statement of policy in the main entrance hall, addressed to the visitor, makes him welcome and directs him how to go about.

C. Just within the entrance door is placed an attractive registration desk with proper equipment and always with fresh flowers in season.

D. On the main floor of the pump room, in full view from the entrance door, is placed a school of instruction. Here are gathered together:

1. A picturized plan of the plant.
2. A table with sample exhibits from each step in the process properly carded.
3. A full sized sectional model of the trickling filters, arranged behind plate glass, showing the nozzle, stone and collecting tile in section.
4. An illustrated flow diagram.
5. An illustrated chart showing the total removals by steps for the preceding year.
6. A complete file of monthly data sheets.
7. Picture groups of 8 or 10 neighboring plants.
8. An attractive four page leaflet. Easy to read. Carefully details the purpose, fundamental principles and general operation step by step in layman's language.

E. All department doors are labeled.

F. In each department is placed a framed statement of the nature of the operation in that room so that the visitor may read his way about as he goes.

G. On Sunday operators change to clean, neat uniforms and are at the disposal of such visitors as care to be shown about.

H. Convenient walks communicating to all parts of the plant invite inspection.

I. The plant is thoroughly cleaned up every Saturday afternoon with careful attention to clean windows, walk-ways, cobwebs, tools in order, etc.

With such an arrangement, if it is a busy morning, a visitor can be greeted in a few minutes and generally directed as to how to find his way about. He, appreciating the situation, finds himself enjoying his own ability to read his way intelligently. He leaves to come again and bring a friend, proud of the plant and ready to defend it. Without these arrangements and left to wander about he finds himself in a hopelessly confused tangle of things he does not understand, and leaves dissatisfied without quite knowing the reason why.

Whenever possible the visitor should be piloted about. This particularly applies to groups, the visit always being preceded by a short lecture in the school which prepares him for a proper understanding of the things he has come to see. The trip should always parallel as closely as possible the sewage flow so as not to confuse the visitor as to the sequence of processes. Such obviously visible improvements—as that across the settling tanks—should be pointed out.

The writer is convinced that no similar effort in any other undertaking about the plant will return such enduring and valuable dividends, to the plant, its trustees, superintendent or to the community served as a well planned visitors program.

Discussion of Sewage Works Problems

D. C. Sperry, *Fayette:* We have had a little trouble with congestion of our sewers, for our flush tanks were not working. We had trouble with waste from restaurants and garages. We just put in 5 new flush tanks, and now we don't have one call to flush out a sewer where we had 20 before. Our 5 flush tanks cost us less than $100 and have given fine satisfaction.

Mr. McMillon, *Mt. Pleasant:* We have had some trouble with our sewers during the last year. Mount Pleasant lies at the fork of two small streams and we have two trunk lines carrying sewage. The Number 2 trunk line had given us a lot of trouble. During a rain the water would come down with such speed and volume that it would knock the

covers off the manholes and the fish, turtles and tadpoles would come up into the disposal plant. The State Department of Health has said a lot about sewage killing fish, but we have some of the happiest looking bullheads you ever saw living at our disposal plant. A couple of weeks ago I was out looking for the cause of the trouble, a thing that I have spent a good deal of time doing during the last year, and I found a hole with a good sized creek running out of it. I got some men to excavate and we followed up the sewer and found a big hole about 18 inches long and the width of the sewer. A piece of tin had been laid over the hole and dirt thrown on it, and the tin had finally broken down. There was a telephone pole set right near by. The telephone company had evidently dug there to place the pole, struck the sewer and broke it. Instead of saying anything about it they just placed the tin over the hole, covered it up and placed the pole a short distance away. I am satisfied that that little trouble has cost us several thousand dollars. So I would suggest if you have trouble to look around the telephone poles.

Mr. McCormick, *Sutherland:* Our trouble is this. We have an Imhoff tank and a trickling filter system. There is considerable sludge that comes off the filters, goes into the creek and gets on the cows' udders and contaminates the milk. We did not know what to do. Since the matter has come before the court the plaintiff has made us a proposition. Before this we could not get her to make any sort of a proposition or submit any plans. Now she has come to use and said that if we will dig down across her pasture 200 rods, and put in a tile drain to carry the effluent away she will be satisfied, and drop the suit. Before that we had offered to buy her land or buy the creek, but she would not consider it. We agreed that we would put in jointed tile so the effluent would be largely dispersed along the way. She insists that we must use solid tile that will not allow for the dispersing of the effluent. We refuse to do this.

Mr. Murphy: What type of filter do you have?

Mr. McCormick: We have 7 feet of crushed rock.

Mr. Murphy: Does this sludge that goes into the creek come from the filter or has sewage been by-passed?

Mr. McCormick: It comes from the filter. Sewage is not by-passed. We have had some trouble with our Imhoff tank getting started, but just before I left our operator reported that he was now getting splendid action.

Question: How often do you flush the rocks in your filter?

Mr. McCormick: Never. To my knowledge they have never been washed.

Mr. Sperry, *Fayette:* We wash ours about four times a year or more if necessary. That might relieve some of the trouble.

Mr. Murphy: That would hardly solve the problem as when the sludge was washed from the rocks it would go into the stream, which is what causes the trouble.

Mr. Wieters: Sutherland has one of the largest creameries in the state, which discharges its waste into the city sewage.

Mr. Rogers, *Newell:* Our plant has been in operation since 1919 and last spring was the first time we washed our filter rocks. We used R.F.C. labor and had the men throw out the rocks. It is about a quarter of a mile from the city water lines to the sewage plant, so we had the fire pump connected to the nearest hydrant and by speeding up the pump got enough pressure to wash the rocks and force them back into the filter bed again. The rocks were only handled once. The action of the bed has been greatly improved, but I have already noticed a brownish coating forming on the rocks down through the first three inches. We were pretty careful to clean out the tile at the bottom of the filter bed and the tile carrying the water out on to the bed.

Mr. Wieters: Did you wash the rocks because they needed it or to put the men to work?

Mr. Rogers: The rocks were covered with this sediment and it was not working as efficiently as we thought it should. There was some ponding and dirt. We have a rather haphazard outfit and have a lot of drainage and storm water to take care of in our sanitary sewers. This makes a lot of dirt that we can not get away from until we can put in adequate storm sewers.

Mr. Dye: A certain amount of coating on the rock is necessary. This is a biological film which contains the bacteria necessary for purification. Slime on the rocks does not necessarily mean a bad condition. However, a dark effluent accompanied by ponding is a condition which needs watching.

Mr. Wieters: The point I wanted to make is that if a trickling filter needs washing there is something wrong with it. It should not be washed. After it is washed, it is a month or six weeks before it begins to function properly. If you have clogging or ponding the condition needs watching. It may be that the Imhoff tank is too heavily loaded. I know of trickling filters that have been in operation 15 or 16 years and have never been washed. Frequent washing indicates a very bad condition.

Mr. Dye: When a large amount of creamery waste is contained in the sewage it increases the B.O.D. load of the filter. At Chicago I talked with an engineer who told me of an experiment tried out recently in Wisconsin. A plant was troubled with clogging and they tried the plan of circulating part of the effluent, about 10 per cent, back into the dosage tank. This seemed to relieve the condition.

Mr. Houser: In regard to the washing of the filter beds at Sutherland, Mr. McCormick, I think that before you became mayor the operator told me that one bed was ponding. That might indicate that the filter was overloaded, and doubtless the creamery wastes would account for that.

Mr. McCormick: I think the filter beds are functioning perfectly. I know that after the sprays start I can walk down to the outlet, a matter of 5 or 6 rods, and in just about that time the water will be coming out.

Mr. Dye: My experience has been that a more even dosage rate produces a better effluent than when the water comes through too quickly. An even flow shows good filter operation and produces a better effluent.

Mr. Sperry: I am glad to hear all this discussion about washing filters. We have always washed ours two or three times a year and I have continued to do it. I don't know whether the operator was advised to wash them when the tank was built or why they did begin to do it.

Mr. Murphy: It may be that your plant is overloaded and it is necessary to wash them that often to prevent ponding.

Mr. Galligan: As I understand Mr. McCormick's problem, he has open to him one of two plans: one is to lay a whole tile drain and conduct the water to a point where it no longer produces a cause for damage; and the other is to lay an open joint tile through the field. It seems to me that still another plan would be to put in a settling tank to clarify the effluent sufficiently so it could be discharged and not create a nuisance. I do not believe that it is possible, considering the type of soil in Iowa, to lay a jointed tile which would not produce a worse condition than you have now.

LABORATORY CONTROL OF SEWAGE TREATMENT PLANTS

By Jack J. Hinman, Jr.

Assoc. Professor of Sanitation, State University of Iowa, Iowa City

It is a little difficult in discussing laboratory control not to get technical. I will discuss the matter of laboratory control in a more elementary way if I can, and take up some considerations that are too often overlooked when the laboratory man is asked to make a judgment on some sort of a problem of a sewage treatment plant.

In my position I am at a good deal of a disadvantage. I have to assume that the samples that come to our laboratory are taken with due knowledge of the needs of the situation, that they are really representative, and that they have come to me in good condition. I know that my fundamental assumptions are often not justified.

To begin with, any laboratory work is dependent upon the character of the sample of material that comes up for examination. It doesn't

matter whether the samples are simply for the Imhoff tank cone test, for the determination of suspended solids, or whether the material is to be used for complicated determinations. If the samples are not right to begin with then the information is wrong all the way through. In other words, the taking of the sample conditions the report, and if the sample is wrong the report is wrong no matter how excellent the work done in the laboratory. In my work I know that people deliberately falsify samples. The result of such falsification is that they get the wrong kind of information in spite of one's best efforts to give them an analysis that is correct. Because we know that samples are often falsified, we have printed at the bottom of our report, "The opinions given in this report are based on the assumption that the samples are collected as specified and shipped exactly as collected."

There are four things in any laboratory work on sewage or water that become very important in the final result.

The first is, how the sample was taken. Was it taken so that it is really representative? If it was not the analysis is worth just the amount of care that was put on that specimen.

Next, was the sample cared for properly between the time it was collected and the time it was brought to the laboratory and examination started?

Third, was the laboratory work properly performed? I feel more certain about that phase as a rule, not only in our own organization but in others as well. In general the men who handle that part of the work have been specially trained.

Fourth, how were the data interpreted? A person's experience and his knowledge of the situation make a tremendous difference as to how well he is qualified to interpret the laboratory findings. Everybody is not familiar with the fact that different laboratory determinations have different degrees of sensitiveness. Most of us know that there are what are known as significant figures, and beyond those the figures are correct within a certain range. Different determinations have different degrees of sensitiveness. The determination of nitrite nitrogen by the sulphanilic acid method gives results that are accurate within .002 part per million. The determination of nitrates is not so accurate. If you get that within .1 part per million you are doing well. People do not realize what these different figures mean and how much they are affected by circumstances. One city council wrote back to us very much disgusted. They said they had no faith in a laboratory that could not get results that checked 100 per cent. The difference was that on one sample we reported .002 part per million of nitrite nitrogen, and on another we reported .003 part per million. We were within the limit of experimental error, even if it had been on the same sample. These people

reached a false conclusion in regard to the laboratory work because they did not understand the meaning which should be attached to these figures.

Going back to the discussion of sampling, let me say that it is quite disappointing to work with some samples I get. It is extremely common for some individual who thinks he has a grievance against a town because his cattle are being bothered or something of the sort, to go out with the family lawyer with three jugs and take a sample. Assuming that he is trying to get a correct sample, he will take one sample up stream, one at the outlet of the sewer, and one downstream. Then these are sent to me marked 1, 2, 3. He tells us nothing about the conditions at the time he took the samples, the flow of the stream, the relative amount of sewage coming from the sewer, or the time of day it was taken. Usually he backs this up with certain official records requesting analyses. A little later I hear that he and his lawyer have come to town to take my deposition. The lawyer asks me a lot of questions to which I must answer, "I don't know." And he asks me if they do not have anyone in the state to do this work who does know. Then I try to explain to him what he has done and how little information can be given on that basis. I'm not willing to swear about the significance of the analyses of any such samples.

The chances are that he got a bad sample. It is most likely he does not understand how stream flow works at a sewage outlet, given a stream with a piece of straight bank. The sewage is warm, as a rule warmer than the stream, so it mixes very little with the stream water. It will flow along the top. Moreover it will hug the bank on which the outlet is located unless there is an abrupt curve to change its course. You would be surprised to know how long it takes to get sewage mixed into the stream. Suppose you take a sample 5 rods or 10 rods down stream, in a stream with straight banks. The sewage slowly fans out into the stream. If he gets a sample along the bank, does he stick him arm way down deep in the water? Probably not. He sees some floating particles and does not go down very deep, but takes the sample from near the top. Now if you need to get a sample from the same point, 5 rods below the sewer outlet, you could take one down deep and get entirely different results. Then he would say our laboratory work is all wrong.

What happens when there is a bend in the stream is more complicated. We have a place in the river at Iowa City which I like to use as an example. There is no sewage discharge at this place, but the river there is a favorite place for canoeing and so is familiar to all the students. Suppose there is a curve in the stream with the sewer outlet on the inner side just before the curve. The sewage in that case will not follow the bank but will go across the stream and hit the other bank, and then probably

come back to the side where it started, but much farther down stream. Another thing is that it is not going to stay on the top. The friction on the bottom of the river will turn it under. If we get a sample below where the current moves across the river we will get water that comes from underneath the sewage outfall.

How would you get a true sample? First you would want to know the discharge. You would then divide the stream into a certain number of sections and take readings and try to weight them and figure your discharge. You will find then, as a result of taking several sewage samples, that at some point, say at a certain depth, a certain distance out from the bank, under a certain degree of flow that you will get a good average. A lot of stream studies are being made by getting such a point, and securing samples at that point. You have no knowledge that this point stays typical but it is lots better than a catch sample.

I want to say a little more about the stream, and the dissolved oxygen it contains. The dissolved oxygen gets in the stream chiefly in two ways. One is that the stream picks up oxygen from the surface of the water. There is a down streaming of oxygen into the liquid. This matter is very well discussed by Adeney, in "Disposal of Sewage by Dilution." With a turbulent stream there is more opportunity for absorption of oxygen. Another source of oxygen is that given off by green plants during the day. We ordinarily say that people give off carbon dioxide and plants give off oxygen. That is not strictly true with regard to these green plants. In respiration we take in oxygen and give off carbon dioxide, and so do plants. But the plant has a function which we do not have, that is during the day and under the influence of the sun and the help of the chlorophyll, the green plant is able to take in carbon dioxide and other simple substances and give off oxygen. When the sunlight is not available to the plant it does not produce oxygen, but it goes on with its respirative activities, takes up oxygen and gives off carbon dioxide. The whole story is based on the proportion between those two activities. During the day time the plant uses the carbon dioxide and other simple substances in forming food, but at night the plant uses oxygen. This is why a lot of algae in a pond may suffocate fish during the night when the oxygen is depleted.

We give a lot of thought to the dissolved oxygen in the water and its effect on the B.O.D. The plants have not yet started early in the morning. Along in the afternoon they have reached their maximum of photo-synthetic activity. In the afternoon the oxygen content of the water is very materially raised above that which is typical of the 24-hour average. If you take a dissolved oxygen sample in the afternoon you will get too good results. If you take samples early in the morning you will get low results. It is not easy to sample water from

a stream and get true results. I have just given you a few points to keep in mind.

As early as 1872 people realized that the ordinary catch sample of sewage was worth nothing. They realized that they had no knowledge that the sample was typical or represented the average. In 1872 the Royal Commission in England made a series of samples on sewage, samples which were collected as composites based on samples during different times of the day, with amounts in proportion to the flow at the time. Perhaps this point should come later when I am discussing analytical procedure. 1872 is way back in analytical work on sewage. The thing which gave sewage work its greatest impetus came along in 1840 when the English government appointed a commission known as the Commission on the State of Health of Large Towns. Prior to that time the sewage system had been largely for storm water, drains and things of that sort. As late as 1850 or 1855 the city of Leeds, England, passed an ordinance providing a fine of 5 pounds for any person putting fecal matter into the sewers. That was typical of ordinances before that time. The reason is that up to that time people had cesspools and privies and sewers were not used as ours are now. The reason that the commission was appointed was that the Thames river got bad and some sewage treatment had to be undertaken.

Water examinations have been going on for a long time. Even the Romans had a rough test for water purity based on the amount of wine necessary to turn the water red. When the work on sewage began in the '70's or before, the workers had the inheritance of the water analysts. Some of these procedures are not well adapted to sewage analysis, but they were taken over and some are still in use. We must keep in mind that these tests can not all be directly compared. The dissolved oxygen, B.O.D., and oxygen consumed, etc., are all things which mean something very definite to the chemist but they can not be directly translated from one to the other. What I want to bring out is the high degree of fluctuation in the character of sewage and the necessity of taking that into account.

The next point is taking care of the sample before the examination begins. I suppose you know that our methods of calculating the numbers of bacteria started in about 1883. It was not until a little later, 1885, 6 or 7, that we knew why water filters worked to an advantage. People began to understand that bacteria multiplied in a water sample after taken, if the sample was allowed to stand. The first article on this is dated 1886. They commenced to understand that a sewage sample changed much faster in its composition than a water sample. The next thing they tried was to stop that change so the sample could be transported. They have been trying ever since and have not suc-

ceeded yet. In 1898 chloroform was tried to stop the change in sewage samples on the way to the laboratory. A lot of work has been done on that one phase alone. Some report that it works fine and others do not find it so. Sulphuric acid, sodium hydroxide, and formaldehyde, have all been tried as preservatives but they all interfere with certain determinations. Today the two outstanding methods of preservation are the addition of enough sulphuric acid to give it an acid reaction equal to 1000 parts per million, and the use of chloroform. Mercuric salts have been tried, but these form a precipitate of the protein material. The latest report published a year ago, after making a survey of the various methods, recommends the use of mercuric salts. In making a report on sewage in Chicago, the workers found that chloroform would do the work in a satisfactory manner. They had to use plenty of it in order to keep the sewage from changing and also keep it on ice. The chloroform used must be washed. Within the last year or two there has been reported in our journal, doubtless many of you read it, an article on the washing of chloroform to get rid of the alcohol. The greatest amount of change in a sewage sample takes place within the first 4–6 hours no matter what material is used to preserve it. As far as I know the best methods are the use of chloroform, low temperatures and transporting the material as quickly as possible. Preserved samples are of no use for bacterial determinations, but of value for certain other determinations.

I realize that I have not given you a balanced discussion on laboratory work on sewage. But I do wish to emphasize the fact that the operator of every sewage treatment plant can make certain simple determinations such as the relative stability test with methylene blue; every plant should make Imhoff cone tests, or rather the Spillner cone; every plant can make determinations of hydrogen-ion activity.

SULPHUR IN SEWAGE

By W. S. Mahlie

Chemist in Charge, Water and Sewage Purification, City of Fort Worth, Texas

Sulphur is perhaps the least understood element and one of the most important factors of sewage disposal problems. It is generally the chief source of odor, either in the sewerage system or the treatment plant.

J. R. Downes [1] states that odors around a sewage plant may be classified as local or migrating and that hydrogen sulphide odors are of the migrating type.

Sulphur in sewage as inorganic sulphate or combined in organic matter may be perfectly inodorous but some time later during treatment or flow through sewers, decomposition may set in and give rise to the obnoxious odors of hydrogen sulphide. Not only are offensive odors given off but the hydrogen sulphide in time attacks the sewers and manholes above the water line thus causing serious damage to the collection system. Persons living near such sewers or their outlets will be annoyed by blackening of the lead compounds in the paint of their houses. Complaints have also been received from blackening of the family silver.

Bach [2] considers sulphur, rather than the nitrogen compounds, as the origin of odors.

Buswell and Elder [3] say: "From the standpoint of odor control, stabilization of the sulphur compounds is particularly important."

Henderson and Haggard [4] report that of the sulphur compounds existing in sewage, 0.000001 p.p.m. hydrogen sulphide, or 0.00000004 p.p.m. of mercaptan can be detected by smell.

Many cases are on record of concrete sewer disintegration and odors. Probably the ones that have received most publicity are the Orange County, Los Angeles County and El Centro County outfalls.[5]

Sulphur in its various forms in sewage, through the agency of bacteria, enzymes, or direct chemical action, undergoes changes depending on the environment and the bacterial action predominating. For instance, under certain conditions approaching anaerobic, hydrogen sulphide will be formed by the reduction of other forms of sulphur, while under aerobic conditions the hydrogen sulphide is changed to sulphates or free sulphur.

The following diagram [6] shows the sulphur cycle graphically.

Rideal says, "During the decomposition of proteins the odors which are given off are not due alone to hydrogen sulphide, the mercaptans, and amines but other substances regarding which we have practically no knowledge." Although Rideal found some methyl mercaptan in the

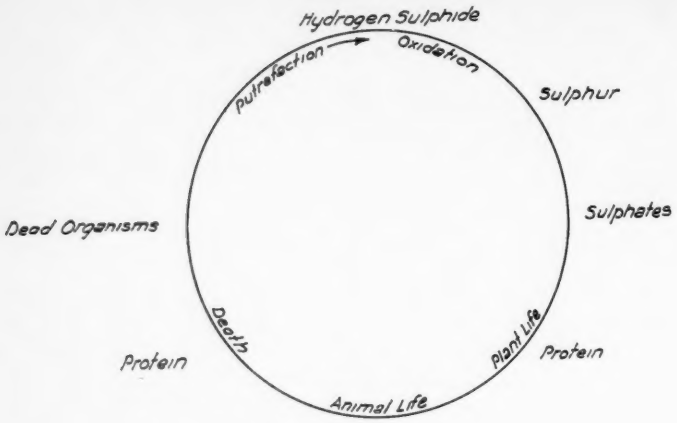

FIG. 1. Sulphur Cycle.

gases from a septic tank, Jesse [7] at the University of Illinois failed to find mercaptans in Illinois septic tanks.

Marshall [6] (page 217) points out that in the anaerobic decomposition of sewage, hydrogen sulphide can be liberated either by splitting of the protein substances or by reduction of sulphates. He considers the latter as the predominating action.

Babbitt [8] states "hydrogen sulphide may be produced apparently in the presence of oxygen, the action which takes place not being thoroughly understood."

While the actual mechanism of the various sulphur changes is quite complex, the entire subject of sulphur in sewage treatment crystallizes itself into the suppression of the formation of hydrogen sulphide and its subsequent removal in the treatment processes.

ORIGIN OF SULPHUR IN SEWAGE

Buswell and Elder [3] point out that sulphur in sewage may originate from three sources: (1) the sulphur present in the water supply which carries the sewage; (2) that from industrial or trade wastes; (3) that from excreta.

Sulphur from Water Supplies exists principally in the form of sulphate combined with sodium, calcium or magnesium or mixtures of these in varying proportions. Rudolfs [9] has shown that the ion in combination with the SO_4 seems to be of some importance in the rate at which decomposition occurs. He found that hydrogen sulphide formation decreased in the following order: ammonium, magnesium, calcium, sodium, aluminum. This would indicate that for the same sulphate content,

softened water (where the sulphate is combined with sodium) would give rise to less odors than the original hard water. Ordinarily the sulphur from water supplies would not vary greatly from time to time.

The following table shows the sulphur content of various water supplies:

TABLE I.—*Sulphur Content of Municipal Water Supplies*

	Number of Supplies	SO_4 P.P.M.	Equivalent as Sulphur
Illinois	14	83	27.7
Ohio	10	91	30.4
Texas	10	78	26.0
California	17	32	10.7
Kansas	3	209	69.7
New York	13	14	4.7
Ft. Worth City Supply		26.9	9.0 Average
" " Private Wells		90.5	30.2 Average of 62

From U.S.G.S. Water Supply Paper 496 (1923).

Sulphur from Trade or Industrial Wastes may be in a great many forms depending on the nature of the waste. The amounts probably would vary considerably from time to time. At Fort Worth we are concerned principally with two kinds of industrial wastes entering our sewers: (1) Packinghouse wastes to the extent of 2 to 2½ m.g. daily and (2) spent acid wastes from washing gasoline at an oil refinery. The packinghouse waste comes from two different plants. Sample A taken 12–15–32 shows 26.9 p.p.m. S and sample B taken 12–15–32 shows 69.8 p.p.m. S.

Sample A is from a plant using the municipal water supply containing 9.0 p.p.m. S, while sample B is from a plant using a well supply containing 177.0 p.p.m. SO_4 or 59.1 p.p.m. S.

The oil refinery wastes consist principally of spent sulphuric acid, an analysis of which is as follows:

	Mg. per liter
Total alkalinity to methyl orange	17150
Alkalinity to phenolphthalein	5750
Total solids	73740
Loss on ignition (corrected for bicarbonates)	7369
Silica	61
Iron and aluminum oxides	36
Specific gravity at 60° F. (Hydrometer)	1.05
Calcium (Ca)	4.3
Magnesium (Mg)	—

	Mg. per liter
Sodium (calculated to combine with Cl-CO_3-HCO_3)	8473
Sulphate (SO_4)	17735
Sulphite (SO_3)	15658
Sulphur in other forms (S)	1549
Chloride (Cl)	900
Carbonate (CO_3)	6899
Bicarbonate (HCO_3)	6893
Lead (Pb)	235

This waste, however, does not enter the sewers in the condition shown by the analysis. It is mixed with varying quantities of other spent water and alkalies.

The mixture as received at our sewers is shown below.

AVERAGE OF 16 SAMPLES TAKEN FROM 12–9–32 TO 3–16–33

	Parts per Million		
	Average	Maximum	Minimum
Total Alkalinity	1,987	17,177	423
Phenol Alkalinity	570	4,980	70
Sulphur as "S"	661	1,394	301

The waste is usually turbid, of a yellow or light brown color and smells strongly of "cracked" petroleum.

Sulphur from Excreta (Urine).[10]—The average amount of urine excreted daily per person is about 1000 to 1200 cc. The total sulphur excretion will average about 2.5 grams equivalent SO_3. The forms in which this is excreted are shown below.

The ratio of nitrogen to sulphur in urine is generally given as 5 to 1. The sulphuric acid excreted by the urine arises principally from the oxidation of protein material within the body, a relatively small amount is due to ingested sulphates.

Sulphur from Excreta—Feces.—Various authorities estimate the amount of feces excreted daily from 110 to 170 grams. Hawk gives a figure of 100 grams based on 1000 tests. According to Cammidge the total amount of sulphur excreted expressed as SO_3 is 0.06 to 0.17 grams daily. Very little information was found to show in what forms the SO_3 existed, but it is assumed that most of it existed in an unaltered condition, i.e., undigested.

Sulphur Content of Foods.—Most of the sulphur in foods exists in the proteins. Proteins are complex mixtures of amino acids and their

Total Sulphur (2.5 gms. as SO_3)

Unoxidized or Neutral Sulphur 5 to 25 per cent of Total	Oxidized or Acid Sulphur 75 to 95 per cent of Total	
Contained in	About 90% is sulphate	
Cystine	of	10% is
Cysteine	Na	ethereal sulphate
Taurine	K	i.e. sulphuric
Hydrogen sulphide	Mg	acid combined
Ethyl sulphide	Ca	with aromatic
Thio cyanates		compounds
Sulphonic acids		such as
Oxyproteic acid		phenol
Alloxyproteic acid		indole
Uroferric acid		skatole
		cresol
		pyrocatechol
		hydroquinol

Arranged from Hawk.

derivatives, and when broken down yield a number of other compounds, the principal ones being the amino acids. Of the common amino acids, cystine is the only one containing sulphur ($C_6H_{12}N_2O_4S_2$), 26.7 per cent S.

Proteins when containing sulphur generally run from 0.3 to 2.4 per cent. It is generally believed that sulphur when present in proteins is present in greatest amount in the cystine, and in this form is rather loosely combined, consequently it is this sulphur which breaks down easily to form H_2S.

Buswell and Elder give some interesting figures by which they show that if the average amount of SO_3 excreted daily per capita is 3.0 grams, this amount when diluted to 100 gallons (the normal sewage flow per capita) will account for only 3 to 4 p.p.m. sulphur.

Unless the sewage contains trade wastes the majority of the sulphur originates in the sulphates of the water supply.

Forms of Sulphur

Sulphur occurs in two general forms in sewage, organic and inorganic.

Buswell and Elder have shown [3] that the inorganic form predominates. The average of seven samples of raw sewage showed that only 1.44 per cent of the total sulphur was present as organic sulphur. These two forms, inorganic and organic, may be present as soluble or insoluble compounds. Here again the soluble forms predominate. A few tests

were made at the Ft. Worth plant to see how much of the sulphur existed as soluble and insoluble forms, with the following results:

TABLE II.—*Soluble and Insoluble Sulphur in Sewage*

		Sulphur* in Solids Per cent	Sulphur in Filtrate P.P.M.
9–27–33	Raw Sewage Solids	0.52	27.1
10–27–33	" " "	0.51	13.6
9–27–33	Imhoff Tank Sludge	0.72	11.5
10–27–33	" " "	0.92	5.8
9–27–33	Digestion Tank Sludge	1.21	4.4
10–28–33	" " "	1.14	9.3
9–27–33	Secondary Tank Sludge	0.69	23.5
10–27–33	" " "	0.79	20.6
9–27–33	Activated Sludge	0.72	27.7
10–27–33	" "	0.91	46.6
9–27–33	Digester Overflow	1.16	13.9
10–27–33	" "	1.20	—
10–28–33	" "	1.22	8.4
11– 1–33	Final Clarifier Sludge	1.01	21.4

* Calculated on dry basis.

Calculating the two samples of raw sewage as containing 284 and 396 p.p.m. suspended matter, respectively, as shown by analysis on these days, the results are:

	Insoluble Sulphur P.P.M.	Soluble Sulphur P.P.M.
9–27–33	1.48	27.1
10–27–33	2.02	13.6

On examination of the results for Imhoff and digested sludge, one is surprised to see the decrease in the soluble and the increase in the insoluble forms. This is also reflected in the tests on the digester overflow liquor. The increase due to concentration of the insoluble by digestion (assuming it is unattacked) would not account for all the increase, and possibly indicates the formation of insoluble forms by combination of H_2S with other constituents, presumably iron, or it may be due to separation of free sulphur as reported by Rudolfs.[11]

We have tried extraction of the dried sludge with carbon bisulphide after first extracting with ether but were not successful in finding free sulphur.

Bacteria Causing Changes in Sulphur Compounds

Gaub [12] determined by special media the approximate number of sulphur-oxidizing and sulphur-reducing bacteria in the sewage treatment plant at Plainfield, New Jersey. He found a progressive increase in the sulphur oxidizers in passing through the plant. He found a decrease in the sulphur-reducing bacteria. However, he found a large increase in this type in passing through the Imhoff tanks, which later was reduced by the sprinkling filters to a very small number.

The results of his tests are as follows:

	H_2S Producers	Sulphur Oxidizers
Raw Sewage	100,000	—
Screen Effluent	1,000	10
Imhoff Effluent	1,000,000	100
Sprinkling Filter Effluent	100	10,000
Final Settling Tank Effluent	1,000	10,000

Later Miss Hotchkiss [13] expanded Gaub's work and divided the sulphur-reducing bacteria into two groups: (1) Those which produced H_2S from proteins. (2) Those which produced H_2S from sulphates. She found no change in the number of protein-reducing forms, but there was an increase in the sulphate reducers.

In the digestion chamber, however, there were marked changes. The sludge itself contained the least number of protein reducers while the liquid between sludge and scum contained the most. Of the sulphate reducers there were more in the sludge, very few in the scum, and larger numbers in the liquid. Of the sulphate producers the least were in the liquid and scum and twice as many in the sludge.

Later tests [14] showed that the H_2S producers in an Imhoff tank were distributed as follows:

Influent	2.51 Per cent
Effluent	3.51 " "
Scum	41.67 " "
Liquid	26.38 " "
Sludge	25.93 " "

Studies of the organisms of a sprinkling filter made on both the sewage and on the bacterial film on the stones showed that the sewage in passing through the filter suffered a reduction in protein reducers and sulphate reducers, while the sulphate producers increased.

Possibly more significance should be attached to the bacteria existing in the film on the rocks than to those in the sewage.

Protein reducers were less in number in the upper portion of the filter than in the lower part. The sulphate reducers first increased in the upper portion, then decreased to a very small number, and then in-

creased again. The most striking, however, was the progressive increase in sulphate producers from top to bottom.

The following table was prepared from Hotchkiss' figures showing the extent to which sulphur-changing bacteria exist in sewage.

TABLE III.—*Percentage of All Bacteria Present in Sewage*

	Protein Reducers	Sulphate Reducers	Sulphate Producers
Imhoff Tank Influent	23.1	1.68	.73
" " Effluent	12.7	1.14	.07
Filter Effluent	7.9	.74	2.96
Final Tank Effluent	6.0	.115	.18

From Paper 172 New Jersey Ag. Exp. Sta.

FORMATION OF HYDROGEN SULPHIDE IN SEWERS

Bowlus and Banta[15] in their studies at Los Angeles, mention the factors influencing the formation of H_2S. (*a*) Sulphate content, (*b*) temperature, (*c*) time and (*d*) concentration or strength.

It is obvious that in order to produce H_2S there must be some source of sulphur, and the more "source material" the greater the potential H_2S formation unless the action is hindered in some way.

Temperature is especially important. Higher temperatures give higher H_2S formation. Figure 2, based on data from Dallas, Texas,

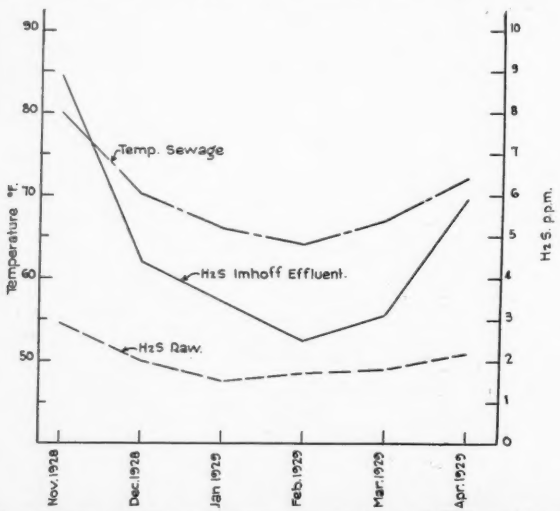

FIG. 2. Dallas, Texas. Relation between temperature and H_2S content of sewage.

shows a nice relationship between the temperature of the sewage and the H_2S content.

Relation between air temperature and H_2S content of the raw sewage is shown in Figure 3 for San Antonio, Texas, but the correlation is not so good as for Dallas.

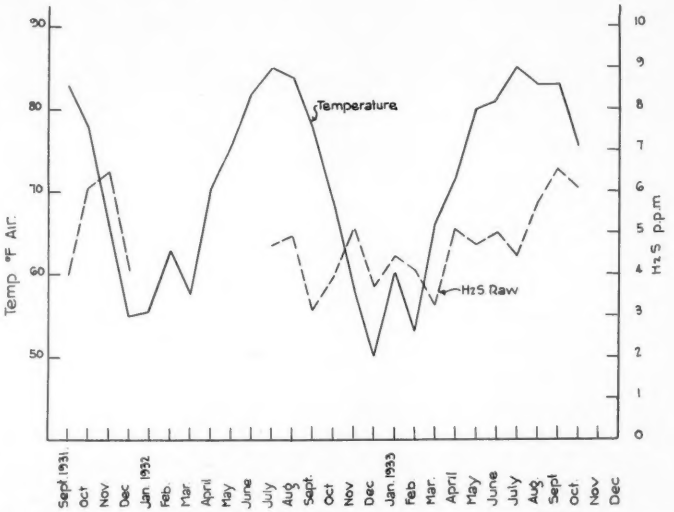

FIG. 3. San Antonio, Texas. Relation between air temperature H_2S in raw sewage.

Figure 4 made from Fort Worth data shows the effect of concentration. While we do not have the B.O.D. results to correspond with the H_2S figures, we know from other tests that they follow the curve for the rate of flow, since the rate of flow is greatly influenced by packinghouse wastes.

Bowlus and Banta [16] attribute the formation of large amounts of H_2S to packinghouse wastes.

Goudey and Reinke [17] state that the H_2S content increases with increased flow of the Orange County outfall sewer. No doubt the strength of the sewage varies in the same general ratio. They show that when fresh sewage is added to a septic or stale sewage the H_2S production immediately increases rapidly. The additional food supply for the bacteria seems to stimulate them to greater activity.

Enslow [18] has shown that the chlorine demand and the H_2S content of sewage are closely related.

At Marlin, Texas, fresh sewage showed a chlorine demand of 5.5 p.p.m. Some distance down the demand was 12 p.p.m. after a well water of high sulphate content entered the sewer. (The demand of the well

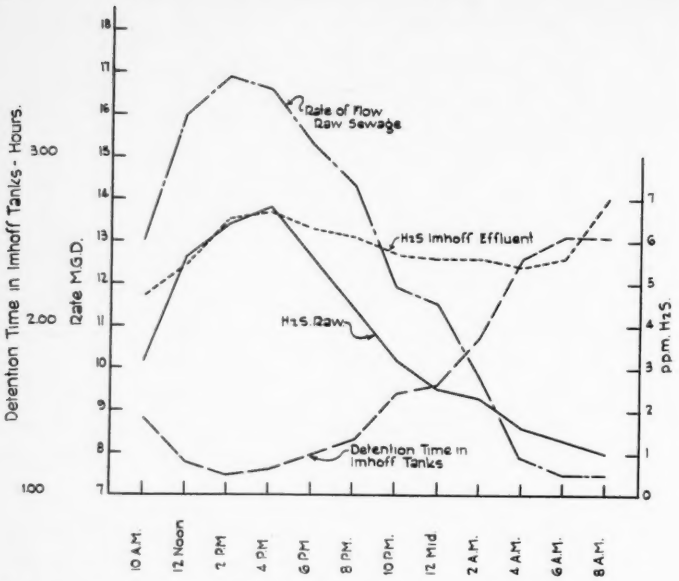

FIG. 4. Fort Worth, Tex. Relation between rate of flow and H_2S content of raw sewage and Imhoff tank effluent. Average of 11 days between 7–21–33 and 9–28–33.

water itself, containing 300 p.p.m. SO_4 was only 2 p.p.m.) Later the same sewage on reaching the plant showed a demand varying from 15 to 35 p.p.m.

Hays [19] found that 2.2 miles above the sewage treatment plant at Cleburne, Texas, the raw sewage averaged 2.5 p.p.m., while at the plant this had increased to 7.7 p.p.m.

Patterson [20] at Newport Beach, California, found that the H_2S in sewage increased from 1.7 to 9.5 p.p.m. in flowing to the plant. After

TABLE IV.—*H_2S Found in Raw Sewage*

	Parts per Million
Havre, Montana	Ave. 3.76
Los Angeles, Orange County outfall	10 to 60
Dallas, Texas	1.0 to 4.2
San Antonio, Texas	3.2 to 6.1
Palo Alto, California	4.0
Cleburne, Texas—2 miles from plant	2.0 to 4.0
" " At plant	6.5 to 9.2
Fort Worth, Texas	2.7 to 10.0
Los Angeles County Sanitation Dist.	Up to 14.0
Newport Beach, Calif.	1.7 to 9.5

a three-foot drop into an equalizing basin the H_2S fell to 6.5 p.p.m. (due to aeration) and in passing through an Imhoff tank it again increased to 7.9 p.p.m.

Hydrogen Sulphide in Imhoff Tanks

Since the Imhoff tank is also a sludge digestion tank, and during digestion H_2S is given off, it is only natural that the flow through a tank tends to wash out some of the H_2S from the digestion compartment.

The condition of the sludge is an important factor. Rudolfs [21] found that digestion takes place in stages which are not very sharply defined and that the acid stage is the earlier stage, characterized by increased intensity of H_2S production. In experimenting on separate sludge digestion, he says, "During the first part of the experiment H_2S was noticeable but decreased later." This same feature has been commented upon by Buswell and Neave [22] as follows: "It should be emphasized, however, that in the acid type of digestion the production and ready evolution of hydrogen sulphide, the formation of volatile free fatty acids, and the liberation of alkyl amines, all contribute toward the offensive character of an 'acid sludge.'"

Enslow [18] in 1926 commented on the increase in chlorine demand of sewage passing through the Imhoff tanks at Dallas and Marlin, Texas. The average of 7 samples at Dallas showed: influent to Imhoff tanks 27.8 p.p.m., effluent 30.0 p.p.m. At Marlin, 6 tests showed: influent 25.7 p.p.m., effluent 91.4. At this time the tanks at these two cities were foaming and the odor of H_2S was very pronounced. At this same period samples from Fort Worth showed: influent 9.4 p.p.m., chlorine demand effluent 8.8 p.p.m. The tanks at Fort Worth were working properly at this time and no odors were being given off by them.

Later data secured from Dallas on actual H_2S content of the influent and effluent showed that there was quite an increase in the passage through the tank, and that the temperature was quite a factor (Figure 2).

For some time we have had exceptionally bad odors at the Fort Worth Imhoff tanks.

Figure 5 shows the relation between the raw and the tank effluent.

At Cleburne, Texas, in a series of tests, Hays found the same condition. Influent to the tanks contained 8.7 p.p.m. H_2S, the effluent 11.5 p.p.m. Later the condition was improved by taking one tank out of service, thus cutting the detention time in half.

R. O. Smith [23] found that with a 45-minute detention period in the Plainfield, New Jersey, tanks the H_2S increased from 0.90 p.p.m. to 1.21.

Referring to Hotchkiss' figures on H_2S producing bacteria previously mentioned, it will be seen that there is an increase of about 40 per cent in going through the tank. Table VI calculated from Hotchkiss' results

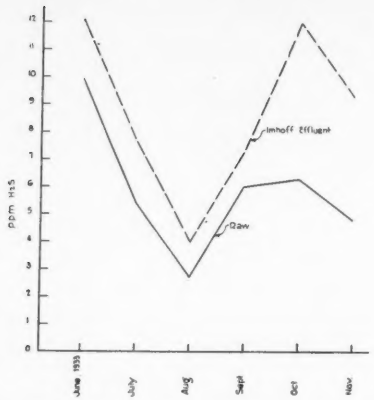

FIG. 5. Fort Worth, Tex. Relation between raw sewage and Imhoff effluent.

TABLE V.—*Distribution of Bacteria in the Digestion Compartment of an Imhoff Tank*

	H_2S Producers Per cent	Albumen Digesters Per cent	Nitrite Producers Per cent	Nitrate Producers Per cent
Scum	33.33	52.29	11.80	2.58
Liquid	30.10	59.55	8.43	1.91
Sludge	53.87	34.24	10.55	1.34

shows that a surprisingly large percentage of the bacteria existing in the digestion compartment of the Imhoff tanks are H_2S producers.

HYDROGEN SULPHIDE IN SEPTIC TANKS

Fales found 15 p.p.m. H_2S in the effluent from a septic tank.

At Virginia Beach, Virginia, Watkins [24] found 2 to 4 p.p.m. H_2S in the influent to a septic tank and 15 p.p.m. in the effluent.

TABLE VI.—*H_2S in Imhoff Tanks and Septic Tank Effluents*

	Parts per Million
Imhoff Tanks	
Dallas, Texas	1.0 to 13.5
Haddonfield, New Jersey	12.0 to 15.0
Newport Beach, Calif.	7.9
Fort Worth, Texas	4.0 to 12.1
Cleburne, Texas	11.5
Independence, Kans.	3.0
Septic Tanks	
Fales (unknown septic tank)	15
Virginia Beach, Va.	15

Hydrogen Sulphide in Sprinkling Filters

The sprinkling filter probably gets blamed unjustly for production of H_2S odors. It should be pointed out that very little H_2S is generated in a filter that is operating properly, although there may be decided odors in the vicinity. The filter sprays are merely liberating the H_2S already present in the sewage.

Barron and Lawrence,[25] and a great many others, have remarked about the liberation of H_2S odors by the sprays.

J. W. Thomson,[26] studying H_2S production at different levels in a filter, found there was a slight increase over that found at the surface, then there was a gradual reduction in passing to the lower levels to the effluent.

R. O. Smith's[23] earlier experiments checked Thomson's results, but later tests showed no increase but a gradual reduction throughout.

Hotchkiss[27] studying the film on rocks of a filter showed that with increasing distances from the surface the H_2S producing bacteria decreased (after an initial increase in protein reducers), and that the sulphur oxidizers increased.

Haenseler-Moore and Gaines[28] showed that *Beggiatoa,* one of the sulphur bacteria, was predominant in summer months and that it was more abundant in the lower levels of the filter than in the upper. They also showed it to be one of the chief binding materials holding the film together.

We have made an experiment at Fort Worth which showed that about

Table VII

Sample Number	Date	Time	H_2S Dosing Tank	H_2S Nozzle Effluent	NH_3 Dosing Tank	NH_3 Nozzle Effluent
1	10–18–33	11:00 A.M.	5.9	3.0		
2	"	1:15 P.M.	2.3	0.8		
3	10–19–33	1:45 P.M.	2.8	1.1		
4	10–20–33	11:00 A.M.	4.3	1.0		
5	"	1:00 P.M.	3.3	0.8		
6	10–22–33	12:45 P.M.	3.9	1.2		
7	10–24–33	11:30 A.M.	4.8	1.3		
8	11–16–33	9:45 A.M.	5.0	0.6	41	41
9	"	12:30 P.M.	3.8	0.7	35	35
Average			4.1	1.2	38	38

Per cent Removal H_2S 70
N as NH_3 0

70 per cent of the H_2S in the influent to the filter is liberated by the spraying action. Pans were placed on top of the rocks, and the sewage was caught in them after passing through the nozzle. Analyses of the sewage in the dosing tank and of the sewage caught in the pans are shown in Table VIII and graphically in Figure 6.

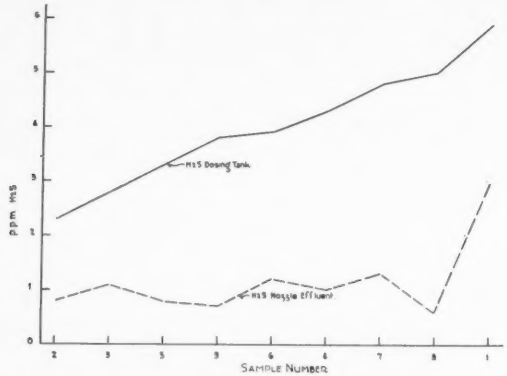

FIG. 6. Fort Worth, Tex. H_2S removed by spray nozzles of sprinkling filter.

HYDROGEN SULPHIDE IN DIGESTION TANKS

The formation of H_2S in separate-digestion tanks is essentially the same as in the digestion compartment of an Imhoff tank or in a septic tank.

Buswell and Elder [22] say: "Of the chemical processes occurring in the digestion tank, protein degradation has long been considered the most important. This fact is attributable to the known offensive nature of the products of putrefaction, notably hydrogen sulphide, volatile amines and mercaptans."

It has been pointed out previously that digestion occurs in stages and that the early, or acid stage, produces the most H_2S. Control of the ratio of digested and fresh solids and maintenance of the proper reaction have been of material benefit in keeping odors at a minimum. Covering the tanks and removing the gas can be so regulated that practically no odors will be liberated. Disposal of the evil smelling overflow liquor, however, does present a problem in H_2S removal.

Buswell and Elder have made experiments to determine whether temperature had any effect on the quantity of H_2S produced. Their curves show that at 37° C. the initial rate of H_2S production was faster than at 22° C., and still slower at 9° C. At the close of the experiment, however, the H_2S in the liquor at 9° C. was greater than that at 22° C. or 37° C., the latter being the least.

In an effort to determine the relation between H_2S and digester temperature, data from the Aurora, Illinois, sewage treatment works have been analyzed and tabulated. The temperatures have been arranged in a series of groups in Table IX.

TABLE VIII.—*H_2S in Digester Gas from Aurora, Ill.*

Range of Temperature Degrees F.	Number of Tests	Average Temperature Degrees F.	Average H_2S in Digester Gas P.P.M.
70 to 75	2	75	391
76 to 80	1	77	456
81 to 85	5	82	556
86 to 90	4	87.5	384
91 to 95	4	92	337
96 to 100	1	100	602

These have been plotted as curves in Figure 7.

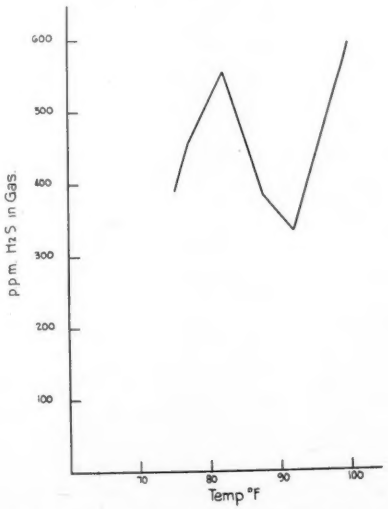

FIG. 7. Aurora, Ill. H_2S in digester gas.

These results indicate that the least production of H_2S occurred at about 92° F. It would be interesting to see if this holds true at other plants.

Enslow [29] says that the H_2S producing bacteria are most active at from 78° to 90° F.

Rudolfs [11] in studying the effect of sea water on sludge digestion found that the sulphates present in the sea water disappeared completely and also part of the sulphates originally present in the sludge mixture. The sulphates were converted to H_2S and there were large quantities of elementary sulphur left behind on the walls of the containers and in the sludge.

Sierp demonstrated and concluded that under strictly anaerobic conditions no H_2S was formed, and that when a small amount of air was added the H_2S forming bacteria were stimulated, and the H_2S oxidizing bacteria retarded.

Rudolfs, however, did find H_2S under strictly anaerobic conditions.

Buswell was unable to detect any free sulphur in his digestion experiments.

Rudolfs in his earlier experiments on separate sludge digestion at 20° C. found that the curve for numbers of H_2S producing bacteria followed the same trend as that for albumen digesting bacteria, and mentions that the inverse relation of these two types of organisms found in an Imhoff tank, was not noted here. Later, experimenting with a large earthen sludge digestion tank covered with a floating cover, in speaking of H_2S producing bacteria, albumen digesters and nitrate reducers, Rudolfs says: "No direct correlation between these, nor any correlation with the method of operation or the course of digestion could be observed."

As far back as 1905 at Columbus, Ohio, experimental studies were conducted to determine how much sulphate reduction took place in a septic tank. The influent to the tanks contained 208 p.p.m. SO_4, while the effluent at the end of 16 hours detention showed 204 p.p.m. SO_4, and after 21 hours detention it showed 201 p.p.m.

Control of Hydrogen Sulphide

The control of H_2S, or strictly speaking, odors, has attracted a great deal of attention and is of prime importance at a sewage disposal plant. To the average person this is of more importance than the quality of effluent produced.

Control measures resolve themselves into two general procedures: (1) to prevent the formation of H_2S, or (2) to remove the H_2S after it is formed.

Preventive measures comprise keeping the sewage moving fast enough to prevent deposits in the lines, a low sulphate content in the water supply, maintaining the sewage at a low temperature, avoiding trade wastes which increase the sulphur content, and keeping the sewage as fresh as possible.

Most of these measures are relatively beyond control of the plant operator and such ideal conditions are not attained in practice, therefore, other means must be employed.

Prevention or removal can be practiced either on the raw sewage, in the treatment plant, or both. The greatest step forward in the prevention of H_2S formation undoubtedly is chlorination of the sewage, either raw or in the course of treatment.

E. B. Phelps [30] states: "Odors from sewage effluents are destroyed through addition of chlorine which combines directly with the odor producing matters such as hydrogen sulphide. The decay and subsequent odor production in sewage may be materially reduced or eliminated if chlorine is applied early in the life of the sewage. Less chlorine than required to produce residual chlorine retards septicization and odor production markedly. Chlorine demand of sewage is influenced by an increase or decrease of the pH value above or below the neutral point of pH 7.0."

It appears that in H_2S control measures by chlorine, the first attempts were made in the treatment plant proper, especially on Imhoff tanks. Later it was found better in some cases to control the condition of the sewage before it reached the treatment plant.

El Centro, California, tried numerous expedients to control the H_2S in an outfall sewer [31] without success. About 1926 chlorine was tried.

One of the outstanding examples is that of the Orange County Outfall Sewer, California,[32] where H_2S without chlorination ran from 10 to 60 p.p.m. and after chlorination it was reduced to 1 p.p.m. and lower.

Enslow says that destroying the H_2S after it is formed is more costly than preventing its formation. The general opinion seems to be that if the H_2S in the raw sewage can be kept below 1 p.p.m. no difficulty will be encountered in subsequent treatment or disposal.

Removal of the H_2S after it is formed can be done either by: (*a*) aeration, (*b*) chemical treatment, (*c*) other methods.

(*a*) *Aeration.*—Concerning aeration, the statements of F. M. Veatch [33] are pertinent. "The application of air for a short period before settling is of benefit . . . and in the case of septic sewage it helps keep down odors by scrubbing out the H_2S and other noxious gases. At Whittier, California, a 15-minute aeration period prior to Imhoff tank treatment has made a noticeable difference in the amount of odor from sprinkling filter sprays. At Dodge City and Abilene, Kansas, H_2S is commonly present during warm weather but after aeration is not noticeable in the tanks or outfall."

It should be cautioned, however, that if aeration is practiced for the removal of H_2S already formed it is important that the expelled gases be collected and deodorized in some manner.

Bach rightfully points out that it is the amount of H_2S that escapes into the air that causes the odor and not the amount remaining in solution.

A sprinkling filter normally liberates the H_2S by spraying. The same is true of any aerating device.

Flint, Michigan, employs a unique method. The Imhoff effluent is aerated in a closed building, the liberated gases are exhausted and forced upwards through the underdrains of a sprinkling filter bed. It is claimed that the filter bed absorbs the odors completely.

In speaking of sulphur bacteria we should remember that the true sulphur bacteria do not form H_2S and cause odors, but actually they utilize the H_2S as a food and store it within their cells as sulphur. Bergey [34] lists some 50 or 60 varieties of such bacteria. Another interesting thing is that many of these bacteria contain a coloring matter, varying from red to purple, which probably accounts for the color sometimes seen on weirs and sprinkling filter stone.

A method for neutralizing odors in gases with chlorine gas has been patented by Henderson and Haggard.[35] This has been applied to odors from incinerators but it is not known whether it has been tried on sewage gases.

Odors in the exhausted gases might also be removed by passage through activated carbon filters or through scrubbers using chemical absorbents.

Sandquist and Foote [36] experimenting at Havre, Montana, found they could reduce the H_2S content of raw sewage 26.5 per cent by agitation with a propeller alone, and with a combination of agitation and chlorine they could reduce it 63 per cent.

At Spring Lake, New Jersey,[37] the entire sewage plant was built underground, the gases exhausted by a fan, and discharged through a high hollow flag pole.

(*b*) *Chemical Treatment.*—In the removal of H_2S by chemical treatment, use is made of the iron compounds, such as ferrous sulphate, ferric sulphate, and ferric chloride either alone or in combination with chlorine, or chlorine alone.

The success of iron treatment seems to depend mainly on the pH value. It is necessary in iron treatment to provide sedimentation for the precipitated sulphur compounds, and to avoid breaking up of the resulting floc by undue agitation, hence it should not be used in a sewer, but should be applied to the influent of a settling tank.

At Haddonfield, New Jersey, Sparmaker and Foreman found that 25 p.p.m. $FeSO_4$ added to the raw sewage entering the Imhoff tank eliminated H_2S odors in the tank effluent.

Downes [38] has reported success in reducing odors from a sprinkling

filter by treating the effluent with iron coagulants and returning the settled sludge to the primary settling tank.

Chlorinated copperas, and ferric chloride mixtures made by passing chlorine solution through iron turnings have been successfully used.

Daniels at Perry, Oklahoma, has had good success using the latter process, when applied to the raw sewage ½ mile ahead of the plant. At Oklahoma City, Scott has been applying this mixture to the influent of Imhoff tanks. At Fort Worth the method was tried without any degree of success, applying the mixture in the sewer about 4½ miles ahead of the plant.

Stevenson [39] has used ferric chloride at Palo Alto, California, with success, and reports that 1 grain per gallon $FeCl_3$ will remove 5.4 p.p.m. H_2S.

Chlorine will unite directly with H_2S gas and precipitate the sulphur according to the equation

$$H_2S + 2\ Cl = 2\ HCl + S$$

It has been found that if the chlorine is applied as a hypochlorite it is more efficient than when used direct.

Viehl [40] reports that in chlorinated sewage at Leipzig, only traces of H_2S could be found.

Dayton, Ohio, has been the recent large outstanding installation employing chlorine and lime for H_2S removal and prevention in Imhoff tanks.

Cohn [41] at Schenectady was one of the first to chlorinate Imhoff tank effluent to keep down H_2S odors on the sprinkling filters.

Cleveland, Ohio, was one of the first cities to chlorinate the effluent from an Imhoff tank to prevent odors in the discharge line, although Ellms and Waring had previously tried it out on a septic tank in 1912 at Cincinnati, using hypochlorite.

At Portsmouth, Ohio, Sheehan chlorinated the influent to the Imhoff tank to prevent H_2S odors.

Hackmaster [42] at Independence, Kansas, found he got the best results by chlorinating the influent rather than the effluent of the Imhoff tanks. Before chlorination the effluent contained 3 p.p.m. H_2S and chlorination reduced it to less than 1 p.p.m.

(*c*) *Other Methods.*—A method thought to be beneficial was the application of a vacuum to the Imhoff tanks, the so-called "de-gasification process." Tests run at Texas A. and M. College by Prof. Steel and P. J. A. Zeller in 1929 indicated a slightly inferior effluent from such a tank. It was found that the H_2S increased 7.0 per cent in passing

through the regular tank and with the de-gasified tank the increase was 9.0 per cent.

At Cleburne, Texas, where the process originated, a series of tests, using the vacuum process as against regular operation of the Imhoff tanks, showed:

		Influent	Effluent
Regular operation	H_2S	8.7	11.5
"Degasified"	H_2S	8.2	10.3

Rudolfs has called attention to the fact that the return of sludge from the final clarifiers of an Imhoff sprinkling filter plant to the raw sewage was of benefit in keeping down odors.

Activated carbon has been used, but it seems that large amounts are necessary.

Toxicity of Hydrogen Sulphide

As far back as 1777, deaths from sewer gas were reported in Paris.[43] In 1806 a committee reported that these deaths were caused by H_2S gas. Analyses later showed the air in these sewers to contain as much as 2.99 per cent H_2S.

Since then numerous cases at other places have been reported.

With the development of separate digestion processes and the collection of gases the hazard of H_2S poisoning has increased.

A number of fatal cases have been reported [44] and it is well to point out that the toxicity of H_2S is quite comparable to hydrocyanic acid.

Method for the Determination of Hydrogen Sulphide in Sewage and Water

Of all the chemical tests used in connection with sewage treatment studies there is probably none which causes more apprehension of the accuracy of the results, than that for hydrogen sulphide.

In the 1925 A. P. H. A. Standard Methods, no method is given for H_2S determination in the section devoted to sewage analysis.

In the 1933 edition, page 60, mention is made that the method given for water is unsatisfactory for sewage analysis.

In "Methods of Chemical Analysis as Applied to Sewage and Sewage Effluents (1929)," the standard of the British Ministry of Health, no mention is made of the H_2S determination.

The general procedure followed by most analysts seems to be to measure the H_2S by means of iodine solution, similar in technique to that employed in water analysis. That this is open to objection has been shown by Hinman,[45] Enslow, Rudolfs, Heukelekian and others.

Generally it not only measures the sulphides, but includes any other substances which will react with iodine.

Nitrites interfere, and no doubt the amount of organic matter present materially affects the results.

A number of the methods are shown in Appendix "A."

The iodimetric methods may be divided into two general plans: (1) in which a known amount of standard iodine solution is used, and the excess titrated with thiosulphate, and (2) in which the iodine solution is added directly to an end point with starch indicator.

Under (1) two procedures may be followed: (*a*) Adding the sample to be tested to the iodine solution. (*b*) Adding the iodine to the sample to be tested.

In the A. P. H. A. Method for Water Analysis, the water is added to the standard iodine solution. This is done no doubt to avoid H_2S losses during shaking.

O. Brunck [46] cautions against the addition of iodine to the sample. The Michigan method for sewage appears to be identical with the A. P. H. A. method for water.

The Kansas and the San Antonio methods are very similar. They both prescribe a slight excess of iodine, acidification and iodine titration. Apparently from the wording of the San Antonio method the excess iodine is not back titrated, while in the Kansas method this is done. The San Antonio procedure can be followed, *i.e.*, the addition of slight excess of iodine, provided there are combined sulphides present, which are liberated by the acid addition. If H_2S only is present, the slight excess of iodine cannot be added, unless it is later neutralized by thiosulphate.

The great difficulty in the method of direct iodine titration is in the variety of red colors formed, and the obscurity of the end point. This is minimized by the addition of excess iodine and subsequent back-titration with thiosulphate.

In methods employing excess iodine additions, the time of contact is important if dealing with sewage. In a test using the Michigan method we obtained the following results:

TABLE IX.—*Effect of Time of Contact on H_2S Determination by Iodine Method*

P.P.M. H_2S

Date	Time	½ Min.	1 Min.	2 Min.	3 Min.	5 Min.	10 Min.
		Raw Sewage					
10–5–33	10:45 A.M.	4.66	4.11	4.56	4.56	4.85	5.50
"	"	2.77	4.01	4.36	4.56	4.95	5.30
"	"	3.91					
"	"	3.76					
	Average	3.78	4.06	4.46	4.56	4.90	5.40
10–9–33		2.62	3.22	3.22	3.47	3.87	4.80
		2.72	3.22	3.32	3.67	3.57	4.61
			2.97		3.42	3.87	4.61
	Average	2.67	3.14	3.27	3.52	3.77	4.67
10–17–33		5.44	5.59	6.24	6.48	7.13	6.73
		5.49	6.08	5.99	5.84	5.99	7.39
			5.69	5.94		6.34	
			6.04				
	Average	5.47	5.85	6.06	6.16	6.49	7.06
Average for 3 days		3.97	4.35	4.60	4.75	5.05	5.71
		Imhoff Tank Effluent					
10–5–33	10:45 A.M.	5.25	5.50	6.43	6.58	6.04	6.58
		5.20	5.50	6.29	6.53	5.94	6.53
						6.19	
						5.99	
	Average	5.23	5.50	6.36	6.56	6.04	6.56
10–9–33		2.42		2.32	3.66	2.62	3.07
		2.57	2.47	2.38		3.37	3.76
		2.28	2.42				
	Average	2.42	2.45	2.35	3.66	3.00	3.42
10–17–33		7.42	7.42	7.39	7.82	7.87	7.87
		7.47	7.82	7.72	8.02		
	Average	7.45	7.62	7.56	7.92	7.87	7.87
Average		5.03	5.19	5.42	6.05	5.64	5.95

This is shown graphically in Figure 8. No doubt the suspended solids absorbed some of the iodine.

R. O. Smith [47] allowed the samples of sewage to settle 30 minutes and then used the decanted supernatant liquid for the test.

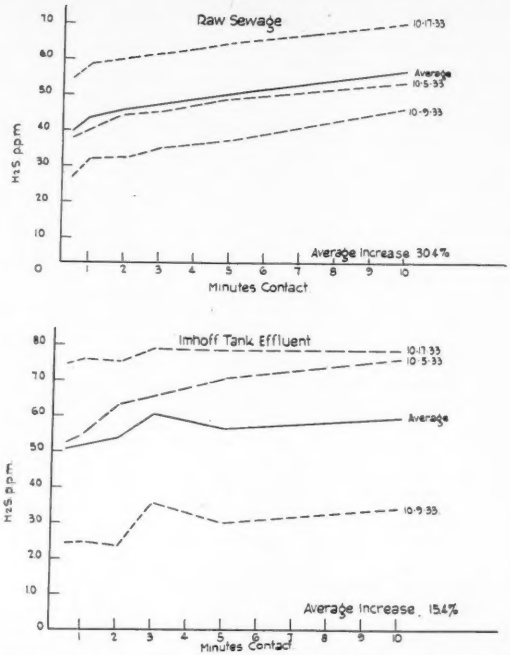

FIG. 8. Effect of time of contact on H_2S determination by iodine method.

Effect of Nitrites.—Any method involving the use of iodine solutions will be inaccurate when used on treated or partially treated effluents, due to the disturbing effect of nitrites in the presence of acid. Nitrous acid ordinarily is a reducing agent, but in contact with potassium iodide it acts as an oxidizing agent, liberating iodine according to the following:

$$2\ KI + 2\ HNO_2 = I_2 + NO + 2\ KOH$$

An experiment was run to prove this. A solution of H_2S in tap water was used. The content of H_2S was determined by the A. P. H. A. method for water. Varying quantities of N as nitrite were added and the H_2S was determined again. Table XI shows the results obtained.

TABLE X

N as Nitrite Added P.P.M.	H_2S Found P.P.M.
None	15.35
1	15.65
2	15.75
5	15.65
10	15.65
25	15.55

In these experiments no acid was added; consequently no nitrous acid was formed. Apparently there is no interference from nitrites unless converted to nitrous acid by the addition of acid, as in the Kansas and San Antonio methods.

It has been shown that nitrites themselves exert no influence unless converted to nitrous acid. We therefore tried out the effect when acidified as in the San Antonio and Kansas methods. The results are graphically shown in Figure 9.

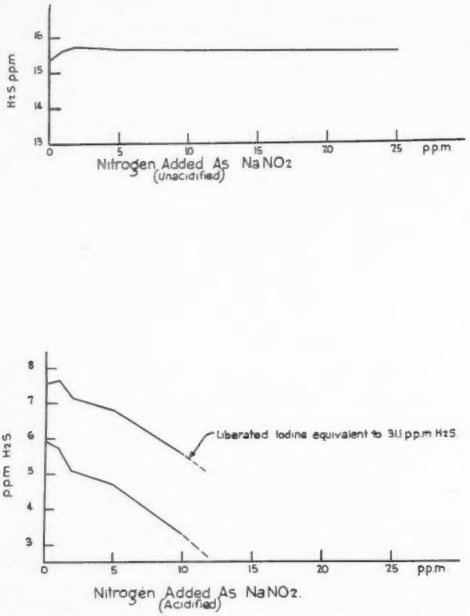

FIG. 9. Effect of nitrite nitrogen on H_2S determination.

N as Nitrite Added P.P.M.	P.P.M. H_2S
None	5.96
1.0	5.75
2.5	5.12
5.0	4.69
10.0	3.30
25.0	Unsatisfactory
None	7.56
1.0	7.67
2.5	7.14
5.0	6.82
10.0	5.54
25.0	Liberated Iodine equiv. 31.10 P.P.M.

Effect of Free Ammonia.—A set of experiments was made to see the effect of free ammonia. A solution of H_2S in tap water, with varying quantities of NH_4OH, was used. The iodine was added to the sample, which was acidified and titrated to a blue end point as in the San Antonio method (first part).

EXPERIMENT *"A"*

N as NH_4OH Added P.P.M.	H_2S Found
None	5.96
1.0	5.64
2.5	5.75
5.0	5.75
10.0	5.64
25.0	5.54

The experiment was again repeated with the following results:

EXPERIMENT *"B"*

N as NH_4OH Added P.P.M.	H_2S Found P.P.M.
None	7.56
1.0	7.45
2.5	7.56
5.0	7.56
10.0	7.56
25.0	7.67

Curves for these experiments are shown in Figure X, *A* and *B*.

Effect of Ammonium Salts.—The same procedure was followed using NH_4Cl as a source of nitrogen.

EXPERIMENT *"C"*

N as NH_4Cl Added P.P.M.	H_2S Found P.P.M.
None	5.96
1.0	5.96
2.5	5.75
5.0	5.86
10.0	5.75
25.0	5.75

EXPERIMENT *"D"*

None	7.56
1.0	7.67
2.5	7.67
5.0	7.67
10.0	7.56
25.0	7.56

Curves are shown in Figure X as *C* and *D*.

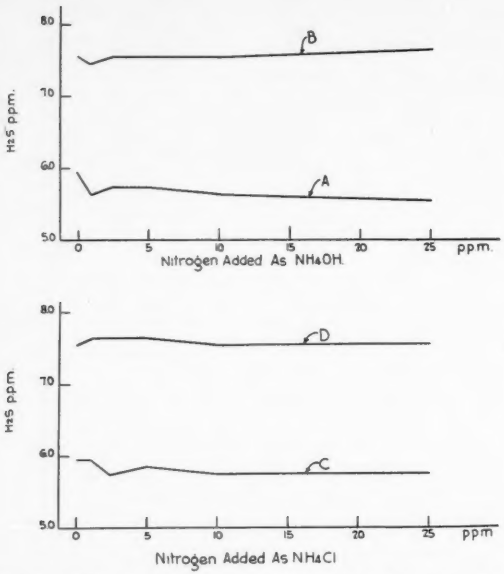

FIG. 10. Effect of ammonium salts on H_2S determination.

These preliminary experiments seem to show that the difficulties with iodometric methods appear to lie in the organic matter content of the sewage, either suspended or soluble, and the nitrites.

In an attempt to compensate for this loss of iodine, the Kansas and San Antonio methods prescribe acidification of a second sample and aeration of volatile iodine consuming constituents (mostly H_2S). The difference between the first (A) and second sample (B) is then considered as hydrogen sulphide. This possibly is not exactly correct as other volatile substances may be swept out which are not hydrogen sulphide.

The California or Salle-Reinke method depends on the formation of a color developed by the action of liberated hydrogen sulphide on an antimony compound (antimony potassium tartrate) the depth of which is proportional to the amount of sulphide present.

Johnson [48] in comparing the antimony tartrate method with the iodine method, points out the difficulty in matching standards when the sewage is colored or contains suspended matter, and gives the accuracy of the California method from 0.25 to 0.50 in sulphide solutions up to 5 p.p.m. H_2S, while for the iodine method, he gives the accuracy as 0.2 p.p.m. The iodine method tended to give higher results. He has worked out a modified method, in which he adds some gelatin to the tartrate solution to prevent the precipitated sulphides from flocculating. He omits the

silica suspension, and prepares his standards from potassium bichromate without cobalt chloride.

Buswell and Elder Modification of Almy's Method

This is a colorimetric method depending upon the formation of methylene blue, by adding solutions of p.-amino dimethyl-aniline hydrochloride and ferric chloride to hydrogen sulphide solutions.

The hydrogen sulphide, and sulphides liberated by acids, are swept out of the sample with a stream of CO_2 gas, and are collected in zinc acetate solution. This zinc acetate solution is then treated with the reagents and the colors compared.

Rudolfs and Baumgartner use this method when the H_2S does not exceed 1 or 2 p.p.m. If more than this they use iodine methods.

Determination of Total Sulphur in Sewage and Sludge

We have endeavored to perfect a method for the determination of total sulphur in sewage. Inasmuch as some of the sulphur compounds in sewage are not completely oxidized it seems that oxidation of the sulphur should be made before precipitation with $BaCl_2$.

We have tried two methods of oxidation: (*a*) bromine water in acid solution; (*b*) $KClO_3$ in acid solution.

The bromine method is as follows: Measure 100 cc. of sewage into a beaker, add 30 c.c. of Br water to the cold solution. Let stand about 5 minutes and then add 5 c.c. concentrated HCl. Digest for 30 minutes, and then precipitate with NH_4OH. Filter, wash well and heat filtrate until odor of ammonia is gone, then precipitate with $BaCl_2$ and finish in the usual manner. Duplicate tests on the same sample showed 34.9 and 35.1 p.p.m. "S."

With sludge samples we allow the bromine to act 15 to 20 minutes, adding more if necessary to maintain a color, and then finish in the usual manner. Duplicate determinations on sludge showed 25.7 and 25.9 p.p.m. sulphur.

In the $KClO_3$ method, 100 c.c. of sewage is measured into a beaker containing 0.25 gram $KClO_3$ crystals. The solution is acidified, allowed to stand 20 to 30 minutes and then digested for an hour. Ammonia precipitation is made as before, the solution filtered and the sulphur precipitated with $BaCl_2$ and finished in the usual manner. One sample showed 38.6 and a check showed 38.7 p.p.m. sulphur. On another sample a check showed 36.4 and 36.0 p.p.m. sulphur.

On a sample of sludge, one set of tests gave 31.6 and 32.3 p.p.m., and on another sample 24.2 and 24.0 p.p.m.

A comparison of the two methods in the same sample showed:

	Bromine Method		$KClO_3$ Method	
Raw Sewage	26.5–26.8	Ave. 26.65	26.8–27.0	Ave. 26.90
Activated Sludge	33.8–33.4	" 33.60	33.6–33.4	" 33.50

It would be expected that the difference between filtered and unfiltered samples of sewage should equal the sulphur found in the dry solids. A test showed as follows:

Raw Sewage—Unfiltered	28.7–28.8	Ave.	28.75	p.p.m.
" " Filtered	27.7–27.5	"	27.60	"
Sulphur in suspended matter			1.15	"
Raw Sewage suspended matter 278 p.p.m. × 0.52%		=	1.45	"
Activated Sludge—Unfiltered	32.0–31.9	Ave.	31.95	p.p.m.
" " Filtered	28.6–28.8	"	28.70	"
Sulphur in suspended matter			3.25	"
Activated Sludge suspended matter 446 p.p.m. × 0.72%		=	3.21	"

This would indicate that in sewage and dilute sludge, the total sulphur obtained by the $KClO_3$ or bromine methods also includes the sulphur present in the suspended matter, provided digestion is made in an excess of oxidizing agents.

Sulphur in Solids of Sludges, Etc.

The sulphur in the solids can be determined by taking the separated dry solids, and analyzing them in identically the same way as in coal or coke by the Eschka mixture method.

APPENDIX "A"

1. Baltimore Method.
2. Kansas University Method.
3. San Antonio, Texas (Same as 2).
4. Michigan Eng. Exp. Station Method.
   Note: The calculations shown in this method are incorrect. 1 c.c. — .025 N thiosulphate equals 0.426 mg. H_2S. The figure .3408 should be changed to read 2.13 to be correct.
5. California Method (Salle and Reinke).
6. Buswell Modification of Almy Method.

METHOD OF DETERMINING H_2S AND SO_2 IN SEWAGE AND SEWAGE EFFLUENTS

Department of Public Works, Bureau of Sewers, Baltimore, Md.

Reagents:

N/100 Iodine solution
N/100 Sodium Thiosulphate solution

Standard ammoniacal zinc solution made by dissolving 3.253 gm. of pure metallic zinc in HCl supersaturating with ammonia and diluting to one liter.

Method.—To 250 c.c. of the sewage sample, in an Erlenmeyer flask, add 10 c.c. of the standard iodine solution and 1 gram of potassium iodide. Bottle is agitated and allowed to stand for a few minutes, and then the excess of iodine is titrated with standard sodium thiosulphate in the presence of starch. This gave both hydrogen sulphide and sulphur dioxide.

To determine the sulphur dioxide alone, it was necessary first to precipitate (in an aliquot) the hydrogen sulphide with the ammoniacal zinc solution. One c.c. of this zinc solution is equivalent to 0.0016 gm. of sulphur or 0.0017 gm. of hydrogen sulphide. Add an excess of the zinc solution (10 c.c.) and then filter off the precipitated zinc sulphide. Sulphuric acid is added to the filtrate, using methyl orange indicator, until all sulphites were converted to bisulphites. Bisulphites are neutral to methyl orange. Ten c.c. of the standard iodine solution were then added and the excess iodine titrated with sodium thiosulphate.

After ascertaining the amount of sulphur dioxide in the sample, this quantity was subtracted from the value found for both H_2S and SO_2 and the H_2S alone was thus determined.

DETERMINATION OF HYDROGEN SULPHIDE IN SEWAGE (DIFFERENTIAL METHOD)

University of Kansas *Bulletin* 16

Reagents:

A. Sodium thiosulphate solution, 0.01 N.

B. Iodine solution, 0.01 N prepared with potassium iodide and standardized against the 0.01 N sodium thiosulphate.

1 c.c. 0.01 N iodine solution = 0.1704 mg. H_2S.

C. Sulphuric acid, 1 vol. conc. acid to 1 vol. water.

D. Starch indicator, freshly prepared.

Procedure.—Collect approximately one liter of the sewage.

A. Measure carefully 200 c.c. into an Erlenmeyer flask. (Throughout this part of the procedure, aeration and consequent loss of H_2S are to be avoided, and mixing of solutions during titration should be done only by a gentle rotary motion.) Add iodine solution until a faint yellow color persists; then add from a pipette 2 c.c. of the diluted H_2SO_4, and if necessary restore the yellow color with more iodine solution. Add 5 c.c. of starch indicator, discharge the blue color by running in sodium thiosulphate solution from a burette, and titrate back to the light blue end-point with iodine solution. Subtract the c.c. of sodium thiosulphate from the c.c. of iodine used, correcting for the titer of the iodine solution, if it is not exactly 0.01 N, and record as "*A.*"

B. Add 3 c.c. of the diluted H_2SO_4 to 300 c.c. of sample, and aerate thoroughly to drive off H_2S. This may be done by drawing air through the sample by means of a water pump, shaking, or pouring repeatedly from one flask to another, *i.e.*, splash

method. Then proceed as under "*A*," except that in this case it is of course not necessary to avoid aeration. Subtract the c.c. of sodium thiosulphate from the c.c. of iodine used, as above, and record as "*B*." Then

$$(A - B) \times 0.852 = \text{p.p.m. of } H_2S.$$

Note: This method is open to the objections that the H_2S may not be entirely removed by the aeration, and that the result is a measure not only of the amount of H_2S present, but also of other volatile reducing agents. However, low values from the first cause are probably within the limits of accuracy to be expected from a field method designed primarily for speed and simplicity, and high ones from the second cause include, besides H_2S, volatile organic substances such as mercaptans and other compounds which are capable of reacting with iodine, but which are, after all, partially responsible for the odor nuisance.

Determination of Hydrogen Sulphide and Hydrous Sulphides in Sewage

Used in San Antonio Sewage Plant Laboratory

Reagents Used:

A. Sodium thiosulphate: .025 N solution.

B. Standard iodine solution: .025 N solution standardized against the .025 N thiosulphate solution.

1 c.c. .025 N iodine solution $\simeq$ .426 mgm. H_2S.

C. Sulphuric acid: 1 vol. conc. acid to 1 vol. water.

D. Starch indicator: freshly prepared.

Procedure.—Collect approximately 1 liter of sewage sample.

A. Measure carefully without aeration 200 c.c. of sample into Erlenmeyer flask. Add standard iodine solution to sample with rotary motion until very slight yellowish appearance due to excess iodine is observed.

Add exactly 2 c.c. of diluted sulphuric acid and mix. Thereafter add starch paste indicator and complete iodine titration. Record the c.c. required.

B. Measure exactly 300 c.c. of sample, add exactly 3 c.c. of dilute acid, mix and aerate thoroughly to drive off the free H_2S gas. This may be done by shaking, blowing with air or pouring from container to container, *i.e.*, splash method.

Measure exactly 200 c.c. of remaining liquid after aeration into Erlenmeyer flask and titrate with standard iodine solution to end-point, with starch iodide, but do not add excess iodine before applying the starch and do not add more acid to this already acidified sample. Record the c.c. of iodine solution required.

$A - B$ in c.c. $\times$ 2.13 = p.p.m. hydrogen sulphide in free state or as sulphides in the sewage.

Note: The probability is that all of the hydrogen sulphide is not completely removed by aeration but the test serves very well as a field test, is rapid and requires no special laboratory equipment nor technique.

Determination of Hydrogen Sulphide

Michigan Engineering Experiment Station *Bulletin* No. 49, page 17

Procedure: (1) Siphon 500 c.c. of the sample into a graduated cylinder.

(2) Pipette 10 c.c. of .025 N iodine solution into each of two Erlenmeyer flasks.

(3) Add 1 gram of potassium iodide to each flask.

(4) To one add 200 c.c. distilled water.

(5) To the other, siphon from the graduate 200 c.c. of the sample.

(6) Titrate both the distilled water blank and the sample with .025 N sodium thiosulphate using starch as an indicator near the end of the titration.

Calculations.—C.c. iodine used for sample minus c.c. used for blank $\times$ 2.13 = p.p.m. H_2S. (Factor .3408 in *Bulletin* is incorrect. W. S. M.)

Note: This method is for both free and combined hydrogen sulphide.

Method for Estimation of Hydrogen Sulphide in Sewage

California Sewage Works Journal, II, No. 1, 1929

Solutions Required:

Antimony potassium tartrate solution, 50 grams per liter.

Hydrochloric acid 1: 1.

Potassium biehromate, 2.5 grams per liter.

Standard cobalt chloride.

24 grams $CoCl_2$ in small quantity H_2O add 100 c.c. HCl—dilute to 1 liter. (Same as used in iron determination Standard Methods.)

SiO_2 suspension, 100 p.p.m.

NaCl crystals.

Standards.—Add 1.05 c.c. $K_2Cr_2O_7$, 2.4 c.c. $CoCl_2$ solution, 10 c.c. of silica suspension for each 1 p.p.m. sulphur. Make to 100 c.c. in 4 oz. bottle. Prepare standards of 0.5, 1.0, 2.0, 3.0 4.0, 5.0 p.p.m. S.

Method for Total Sulphur.—Add 1 c.c. potassium antimony tartrate and ½ gram NaCl crystals to sample bottle. Fill to 100 c.c. with water or sewage. After mixing, add 1 c.c. HCl solution, compare—run blank.

Volatile Sulphur.—Aerate sample for at least one minute. Run as before, subtract from previous figure.

Caution!—Do not add HCl until antimony solution has been mixed.

Buswell Modification of Almy Method

Journal American Chemical Society, **47,** 1381 (May, 1925)

A. Standard hydrogen sulphide solution for preparing standards.

B. p-amino dimethylaniline hydrochloride 0.04 gram per 100 c.c. 1: 1 hydrochloric acid.

C. Ferric chloride, 0.02 M.

D. Zinc acetate, 0.6 per cent.

Standards are prepared in 50 c.c. Nessler tubes by adding 5 c.c. of solution "*B*" and 1 c.c. of "*C*" to 44 c.c. distilled water containing a known quantity of hydrogen sulphide (*A*) per 50 c.c. tube.

The standards range from 0.001 to 0.050 mg. hydrogen sulphide per 50 c.c. tube.

To one liter samples which are used for determining H_2S in sewage, 50 c.c. of concentrated HCl are added, and the flask containing the sample is aspirated with carbon dioxide for 30 minutes. The H_2S is collected in 0.6 per cent zinc acetate and determined by adding the reagents in proportions as given for preparing standards.

For quick approximate estimation of H_2S, 10 c.c. of sewage may be treated directly with 5 drops of the p-amino dimethylaniline reagent and one drop of ferric chloride solution.

More details concerning this test can be found by consulting the original article by Almy.

METHOD OF SULPHIDE DETERMINATION

Rudolfs and Baumgartner, *Ind. and Eng. Chemistry*, **24**, 1152 (1932)

The one method, that of Almy [1] modified by Buswell [3] for sewage analysis, was used when the hydrogen sulphide content did not exceed one or two p.p.m. The other method employed was the one in which hydrogen sulphide is allowed to react with standardized iodine solution. In both cases the hydrogen sulphide was driven out of the sewage by carbon dioxide to remove the interfering substances. This was accomplished by constructing an aerator from large-bore glass tubing. An alundum filter disk was fastened in the lower end of the tube. Carbon dioxide, led into the cylindrical chamber under this plate through a tube, diffused through the plate, giving an even flow of fine bubbles up through the whole of the liquid. A tube at the top of the aerator led the gases off to a ten bulb absorption tube where hydrogen sulphide was absorbed in either zinc acetate solution or allowed to react with standard iodine solution depending on the method used.

In determining total sulphides, concentrated hydrochloric acid was added to the sample being aerated; this accelerated the removal of hydrogen sulphide from the liquid and also decomposed the combined sulphides. One half hour to one hour aeration, depending on the amount of sulphide present sufficed to drive out the sulphides from the sample. When hydrochloric acid was not added, as would be the case where only volatile sulphide or free hydrogen sulphide was determined, longer aeration periods were required. Even then, it could not be ascertained that all the hydrogen sulphide was removed, although the amount remaining would be negligible. This would tend to make the determinations of combined sulphides slightly higher than they were actually.

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Industrial Wastes

MILK WASTE TREATMENT STUDIES IN WISCONSIN

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One of the most common of all industrial waste disposal problems is that occasioned by milk wastes. Washings and other liquid wastes from dairies, cheese factories, creameries, and other plants producing milk products discharged untreated into small streams or drainage ditches have caused numerous complaints of the resulting unsanitary conditions. The wastes decompose, producing foul odors, and dissolved oxygen in the waters of small streams receiving the wastes may be depleted with the resulting destruction of fish and other normal aquatic life. Streams affording ample dilution for the wastes show rapid recovery from such pollution, but small streams virtually become open sewers, particularly during summer months. Abatement of nuisances and gross stream pollution caused by milk wastes has been increasingly insisted upon by the public.

Milk wastes have been the source of serious operating difficulties at sewage treatment works when discharged untreated into municipal sewer systems, occasionally causing the partial or complete failure of plants built to treat the sewage. Such situations are generally encountered in small communities where the quantity of milk wastes is relatively large compared with the total amount of sewage treated.

Development of efficient and economical methods for treatment to secure inoffensive disposal of milk plant wastes, and practical application of such methods in the solution of pollution problems, are objectives of a program being carried out in Wisconsin. An experimental station was established by the Sanitary Engineering Division of the State Board of Health at De Forest in 1927, and studies of various processes of milk waste treatment were conducted until the fall of 1930. Results of the early work have been published.[1] The treatment processes originally considered were activated sludge, chemical precipitation and several types of trickling filters; and chlorination studies were also conducted. It was concluded from this work that treatment of the milk wastes was best accomplished by trickling filters. Based upon data obtained at

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De Forest, as well as the results obtained by Levine,[2, 3, 4] Kimberly,[5] Slaughter,[6] Eldridge,[7, 8] and others, demonstration plants were designed and installed at various types of milk plants where nuisances and serious pollution problems existed. Studies described in this report were made at these plants.

Description of Plants

The first general plan suggested by the State Board of Health included a rectangular flow-equalizing tank to receive the pollutional wastes from the milk plant. This tank had capacity enough to take practically the entire day's flow. The object was to provide storage capacity sufficient to permit a uniform filter dosage rate over a period in excess of 8 to 12 hours, the normal operating day at milk plants. The waste was pumped from the flow-equalizing tank to an elevated dosage tank by means of a float-controlled centrifugal pump. From the dosage tank the waste discharged by gravity through a system of pipes or open channels to a battery of tip troughs above distribution boards, which spread the waste over the surface of the filter. The filter unit was rectangular in shape and eight feet in depth. The effluent from the filter was either discharged into a city sewer, or into a stream after passing through a secondary settling tank for removal of solids sloughing off the filter rock. The filter was housed, the roof being sufficiently high above the surface to allow a man to carry on the necessary cleaning operations. Figures 1 and 2 show views of installations of this type of plant.

Fig. 1. Full-scale waste treatment plants of the trickling filter type, dosed by tipping troughs, and housed as shown above, have been installed at cheese factories, condenseries, a creamery, a bottling plant and a receiving station following experiments at De Forest, Wisconsin.

FIG. 2. Milk waste treatment plant at a Wisconsin creamery.

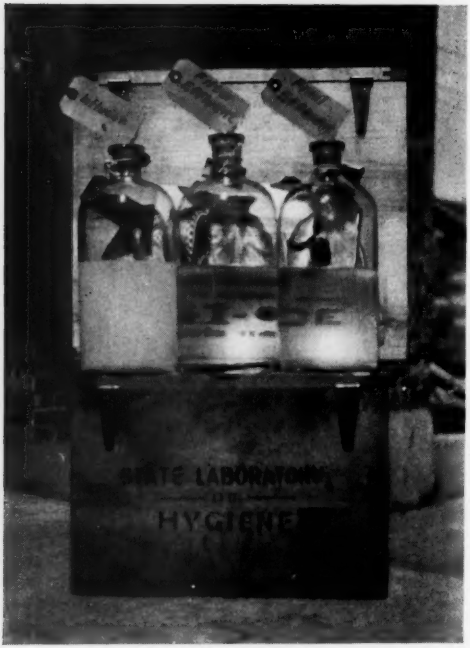

FIG. 3. Milk wastes before and after treatment at the plant shown in Figure 2.

The studies made during 1930 and 1931 indicated that, while in general this type of plant would give satisfactory results (see Figure 3), several changes could advantageously be made. Data showed that the frequency of filter dosage by the tip trough was higher than considered desirable for greatest efficiency. At dosage rates of 2 million gallons per acre per day, the average time between successive applications of waste was about once every minute. The need became apparent for some method of providing better adjustment of the dosing cycle. The studies further indicated that slime growths resulted in unequal distribution.

During 1931, the engineering staff of the Carnation Milk Company, working in conjunction with the State Board of Health engineers, designed a disposal plant for the company's condensery at Chilton, Wisconsin. This design differed from the original described above in that the filter was circular rather than rectangular; the waste being applied by means of a rotary distributor instead of tip troughs. The filter unit was not covered, the walls consisting of a series of pipe uprights, imbedded in a concrete curb, supporting ¾ in. poultry wire strengthened by fence wire. (See Figures 4, 5 and 6.) The waste from the condensery flows to a storage tank and is then pumped to the filter unit by means of a sump pump, the pump being controlled by a time switch. While the dosage period can be readily varied, the filter is generally operated on a cycle of seven minutes dosage and an eight minute rest period. The filter effluent discharges through a secondary settling tank into the nearby stream.

The Chilton plant was studied in considerable detail under summer and winter operating conditions. From observations and analytical results this type of plant is believed to be more satisfactory than the earlier type. A general plan of the circular type of trickling filter unit is shown in Figure 4, together with views of the Chilton installation in Figures 5 and 6.

Method of Conducting Studies

In general, four complete runs were made at each plant during periodic visits, an entire operating day being devoted to each run. The data obtained during these tests consisted of records of the total quantity of waste discharged, filter dosage rates in million gallons per acre per day, together with composite samples of the filter influent, filter effluent, and the secondary settling tank effluent. The filter dosage rates were determined by installing a triangular weir in the channel leading to the secondary settling tank, and at definite intervals the head on the weir was measured. At the beginning of each day's run, the dosage rate was adjusted and kept constant. The length of runs made varied to some ex-

tent, but, in general, were from six to ten hours in length. Composite samples were collected at half hour intervals throughout the duration of the run. Determinations were made of pH values at time of collection. The dilutions for five-day B.O.D. determinations were set up in the field;

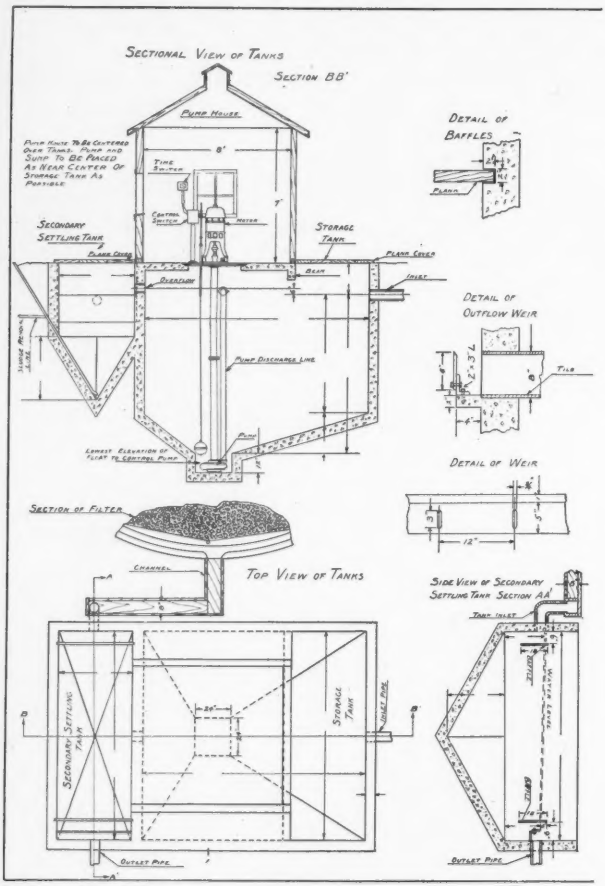

FIG. 4A

FIG. 4. Plan showing general details of milk waste treatment system advocated as a result of recent studies.

and the composite samples were then chloroformed and sent to the State Laboratory of Hygiene at Madison for chemical analyses. Procedures of analysis followed were those set forth in Standard Methods.[9]

The total quantity of waste discharged on a given day was determined by measuring the amount of waste passing through the filter during the test, making correction for any increase or decrease in wastes present in the storage tank at the end of the run.

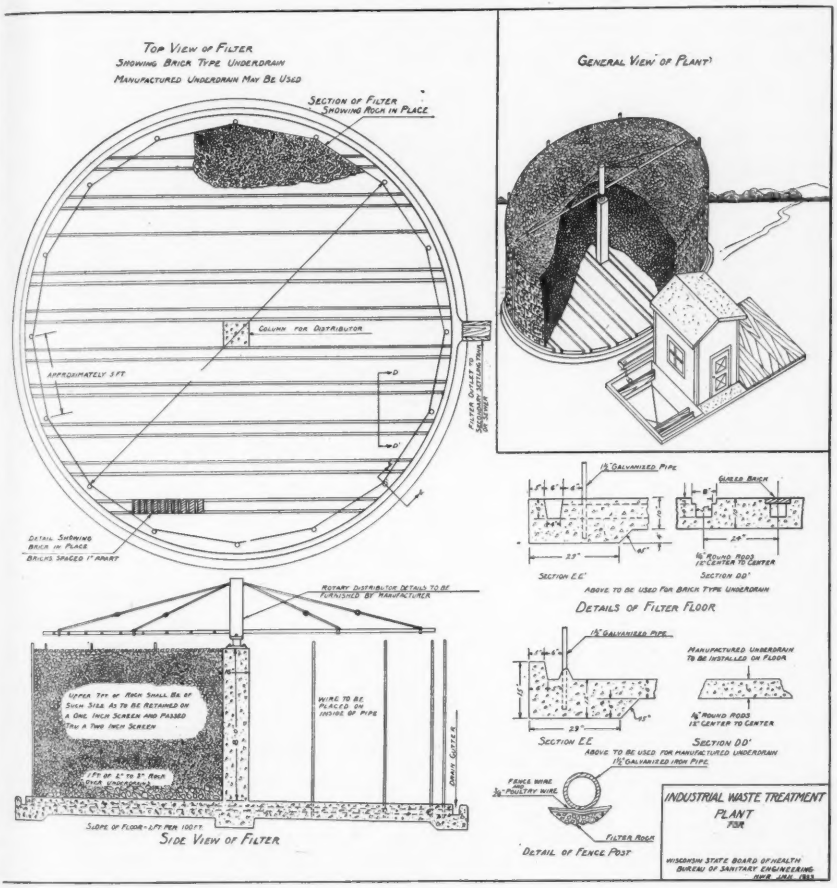
Top View of Filter
Showing Brick Type Underdrain
Manufactured Underdrain May Be Used
Section of Filter Showing Rock in Place
Column for Distributor
Approximately 3 Ft.
Detail Showing Brick in Place
Bricks Spaced 1" Apart
Rotary Distributor Details to be Furnished by Manufacturer
Upper Ten of Rock Shall Be of Such Size As to Be Retained on a One Inch Screen and Passed Thru a Two Inch Screen
1 Ft. of 1" to 2" Rock over Underdrain
Wire to be Placed on Inside of Pipe
Drain Gutter
Slope of Floor - 1/4 in per 100 ft.
Side View of Filter
General View of Plant
1½" Galvanized Pipe
Glazed Brick
Section E.E'
Section D.D'
Above to be used for Brick Type Underdrain
Details of Filter Floor
Manufactured Underdrain to be Installed on Floor
Above to be used for Manufactured Underdrain
Fence Wire and ½" Poultry Wire
Filter Rock
Detail of Fence Post
Industrial Waste Treatment Plant
For
Wisconsin State Board of Health
Bureau of Sanitary Engineering

FIG. 4B

QUANTITY AND POLLUTIONAL STRENGTH OF MILK WASTES

Knowledge of the quantity and strength of milk wastes is fundamental to the proper design of a treatment plant. The data given in

Table I summarize a number of measurements made of the pollutional wastes produced by various milk plants. Included in these pollutional wastes are can, floor, utensil and equipment washings. In considering these data one should keep in mind that one purpose of these studies was to furnish general or average values for certain types of plants, and also to show deviations that might be expected between individual plants

FIG. 5. View of a milk waste treatment plant. System following the general plan shown in Figure 4 at a large condensery in Wisconsin. The flow equalizing tank and pump house are shown in the foreground.

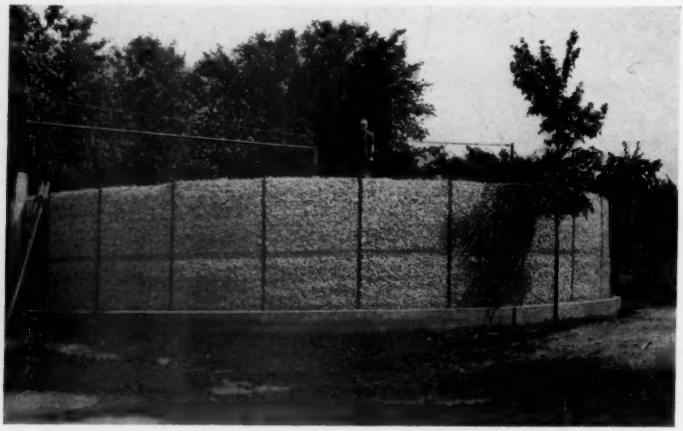

FIG. 6. Trickling filter unit showing overhead feed pipe to the rotating distributor. The dosing system is regulated by a time switch.

classified as to manufacturing process. Before designing a treatment system, waste flow data are considered essential. The fact that the quantity of waste discharged by similar types of milk plants varies over wide ranges is shown by the maximum and minimum values for condenseries given in Table I. It will be noted that the unit volume of

TABLE I.—*Quantity and Pollutional Strength of Waste from Various Types of Milk Plants*

Type of Milk Plant	Gallons of Waste per 1000 pounds of Milk Intake			Population Equivalent per 1000 Pounds of Milk Intake		
	Average	Maximum	Minimum	Average	Maximum	Minimum
Condensery	140	250	70	4.8	7.5	2.7
Cheese Factory	170			16.1	25.2	10.2
Receiving Station	170			2.8	3.9	1.6
Bottling Works	250			4.7	5.2	4.3

Note: The data for condenseries are on a total of 13 runs made on three plants. The data for the cheese factories, receiving station and bottling works are averages of three or more tests on a single plant.

waste produced at these plants varied between 70 and 250 gallons per 1,000 pounds of milk intake.

The data obtained for cheese factories, receiving stations and bottling plants is also included in Table I. Further studies may somewhat modify these values, although they are in fair agreement with similar data obtained in Michigan by Eldridge.[10]

In investigations of stream pollution, it is usually found advantageous to convert the data on the oxygen demand of industrial wastes into "population equivalents" in order to reduce the polluting ability of the wastes and that of the domestic sewage to a common, readily understandable basis. These population equivalents are obtained by dividing the total daily oxygen requirement of the waste under consideration by the daily per capita oxygen demand of domestic sewage. On the basis of a five-day biochemical oxygen demand, this amounts to 0.167 lb. of oxygen. Total oxygen requirements may then be converted into population equivalents per unit of raw material or finished product. In Table I the unit taken for the various types of dairy product establishments in presenting population equivalents is 1,000 lb. of milk taken in per day, oxygen demands being based on composite samples of waste discharged throughout a working day.

It will be noted from the average, maximum and minimum population equivalents presented in Table I that a considerable variation occurs in

the pollution loadings occasioned by plants of the same type as well as those of different types. The greatest loadings are occasioned by cheese factory waste, due to high oxygen requirements of cheese washings and spilled whey. The average population equivalent for the factory studied was 16.1 per 1,000 pounds of milk intake. The data obtained showed that the least pollution loading was occasioned by wastes from the receiving station, which had an average population equivalent of 2.8 per 1,000 pounds of milk intake.

In connection with condenseries the question generally arises as to the quantity and pollutional character of the condensing water used in the concentrating pans. This pan water is usually bypassed around the waste treatment plants. In most cases this water is discharged into municipal storm sewers or directly into the nearest water course. During the past two summers a series of tests was made at several typical condenseries to provide information on pollution loadings occasioned by the pan water.

Data presented in Table II were obtained at a plant where a stilling basin and measuring weir were installed, providing excellent facilities

TABLE II.—*Results of Pan Water Studies at a Typical Condensery*

Temp. °C.	pH	Dissolved Oxygen	5 Day B.O.D.	Oxygen Consumed	Total Solids	Total Volatile Solids	Soluble Solids	Volatile Suspended Solids	Total Gallons of Pan Water	Pounds of Milk Concentrated	Pounds of Milk Concentrated per Pan Hour	Gals. Pan Water per 1000 Lbs. Milk Condensed
35	7.5	4.3	27	50	428	204	418	10	24,800	15,700	7,000	1,580
35	7.5	4.1	17	30	432	140	412	18	23,600	15,900	7,400	1,420

for measuring flows. Analytical data given in the table were obtained from composite samples that were collected proportionate to the flow.

Results indicate an average of about 1,500 gallons of pan water used per 1,000 pounds of milk condensed. This average value, as well as the five-day B.O.D. of about 22 p.p.m. for the pan water, was verified by the studies at two other condenseries. From the information obtained it seems fair to conclude that unless limited dilutions are afforded, or very unusual conditions exist, the condensing water can be discharged directly into a creek or stream.

PLANT OPERATION

During these studies careful observations were made of time necessary to keep the waste treatment plants in satisfactory operating con-

dition. In general it was concluded that two inspections daily were sufficient to insure good operation. The routine of these inspections consists in oiling the pump and removing any material having a tendency to clog the orifices of the distributor or system for applying waste to the filter. The time required per inspection varied from ten to twenty minutes. In the majority of cases one employee was responsible for the operation of the treatment plant, the inspection being a part of his daily routine.

An undesirable characteristic of the original design was the accumulation of slime and scum on the tip trough and distribution boards; if this material was not removed periodically the distribution of the waste over the filter would be reduced to the point where in some cases only about a third of the surface area of the filter received waste. At several plants the rate of growth of this material was sufficient to affect distribution in about two days. In order to keep the distribution boards in good condition it was necessary to remove this growth with a squegee two or three times a week. This operation is entirely eliminated by the use of the rotary distributor.

Cleaning of the storage or flow equalizing tank and secondary settling tank are essential for good operation. A white curdy sludge having the appearance of sour milk was observed in the storage or flow-equalizing tanks in the majority of the plants. Sand and dirt washed from cans and floors is a part of this sludge accumulating in these tanks. If the sludge is allowed to remain in the tank indefinitely, decomposition occurs producing disagreeable odors. The time required for this material to become offensive depends primarily on the season; in the majority of cases a thorough cleaning of this tank three or four times a year is sufficient, along with periodic washing down of the walls with a hose. Experience indicates that two men can thoroughly clean a storage tank, that has not been neglected, in from three to five hours.

In the existing plants the slough-off from the filter, which is deposited in the secondary settling tank, is pumped out or is removed manually. In the design shown in Figure 4, the bottom of this tank has been sloped so that the deposited material will flow to the apex of the hopper from which it can be removed by a pump thus materially reducing the time and effort required to clean this portion of the plant. Experience would indicate that cleaning this tank once a month is generally sufficient.

Filter Loadings and Efficiencies

To date a large number of filter loading tests have been made at nine full-scale plants. These plants take care of wastes fairly representative of those produced by cheese factories, condenseries, receiving stations, bottling plants and creameries. It is believed, therefore, that the vari-

ety of conditions covered warrants drawing certain general conclusions relative to filter loadings and efficiencies.

A detailed study of the data obtained was made in an attempt to secure some basic values to be used in the design of filters for milk waste treatment. It is generally considered that the efficiency of a given filter depends to a large extent on the rate of application of the waste, or the dosage rate, in gallons per unit of surface area per day and the quantity of organic material applied per unit of filter volume per day. The first step, therefore, was to ascertain the relationship between the 5-day B.O.D. removals and the dosage rate in gallons per acre per day. The data indicated that as the dosage rate was increased the efficiency of the filter decreased; a value of about two million gallons per acre per day appeared to be about the maximum allowable dosage rate although in some cases fair B.O.D. reductions occurred at higher rates. The design of a filter unit on the basis of dosage rate only may be a reasonably satisfactory method in cases where the wastes to be treated are rather uniform in character. The strengths, however, of the wastes produced by various types of milk plants vary over large ranges and any design based only on dosage rates would not be satisfactory in cases of this kind.

A careful analysis of the data was made to show the relationship between filter loadings and efficiencies. These studies were made using results of analyses including 5-day B.O.D.'s total volatile solids and total organic nitrogen. The results in each case definitely indicated that as the filter loading increased the efficiency decreased. From these results, and in view of the fact that the 5-day B.O.D. is generally accepted as the most satisfactory single test to show the pollutional character of a sewage or industrial waste, it was decided to select the 5-day B.O.D. loading as a basis for determining filter size. Figure 7 shows the relationship between the efficiency of the filter, expressed as per cent 5-day B.O.D. reduction, and the filter loading in terms of cubic feet of filter volume per pound of five-day B.O.D. per twenty-four hours.

Referring to Figure 7 it will be noted that when the unit capacity of the filter is between 70 and 80 cubic feet per pound of 5-day B.O.D. per 24 hours, an efficiency of 80 per cent or more can be reasonably expected. Above a unit value of 80 cubic feet it will be noted that the increase in efficiency is relatively small for a considerable increase in filter capacity. Accordingly the basis of design for milk waste filters recommended is 80 cubic feet of rock per pound of five-day B.O.D. applied per day, the filter depths ranging from six to eight feet and the size of crushed rock or stone being 1 to 2 inches. Crushed granite, quartzite or other material relatively resistant to wastes having a pH value as low as 5.0 should be provided in the construction of filters receiving cheese factory wastes.

The results of filter loading studies conducted during 1930 and 1931 were embodied in milk waste treatment plants constructed during the past two years. As the plant installed at Chilton, Wisconsin, is one of these more recent plants, and as it has been built most nearly in accord with the general plan shown in Figure 4, and now recommended, a brief discussion of results obtained will be of interest.

The Carnation Milk Company condensery at Chilton produces only condensed milk. The average milk intake during the period these tests were made was about 163,000 pounds daily. All cooling water and the condenser water used in the concentrating pan is discharged directly into a creek; all other wastes pass through the treatment plant. The filter, which is 46 feet in diameter and 8 feet deep, is shown in Figure 5. On

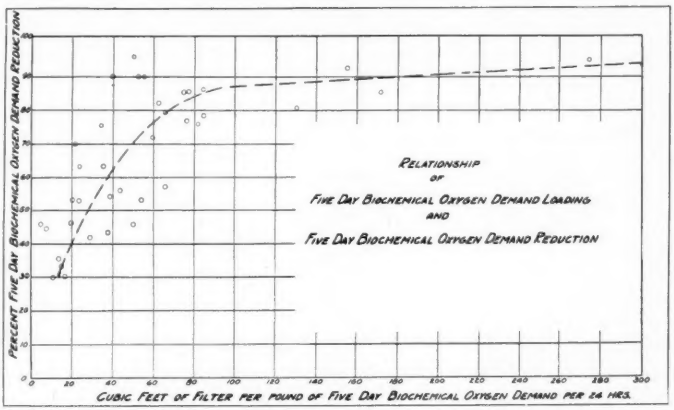
Percent Five Day Biochemical Oxygen Demand Reduction
Relationship
of
Five Day Biochemical Oxygen Demand Loading
and
Five Day Biochemical Oxygen Demand Reduction
Cubic Feet of Filter per pound of Five Day Biochemical Oxygen Demand per 24 Hrs.

FIG. 7. Chart showing results of filter loading studies made at milk waste treatment plants in Wisconsin.

the basis of 80 cubic feet of filter volume per pound of five-day B.O.D. per 24 hours the filter provided excess capacity at the time of the tests. This unit capacity averaged 96 cubic feet. The maximum milk intake at this plant, however, is about 200,000 pounds per day. For this milk intake the filter size would be approximately 80 cubic feet per pound of five-day B.O.D. per twenty-four hours.

The data given in Table III represent an average of a series of six complete tests made on consecutive days at the Chilton plant. The degree to which the removal or modification of pollutional constituents has progressed in passing through the filter is indicated by the complete absence of sugar in the effluent, the presence of nitrate nitrogen, the decrease in the free ammonia content of the waste and the 86 per cent reduction in the five-day B.O.D. of the waste.

TABLE III.—*Results of Milk Waste Treatment Plant Studies at a Condensery* *

	Filter Influent	Filter Effluent	Final Effluent
Dosage Rate, M.G. per Acre per Day	0.75	——	——
Dosage Rate in Cu. Ft. of Filter per lb. of 5-day B.O.D. per 24 hours	96	——	——
Filter Dosage Cycle in Minutes			
Pump On	8	——	——
Pump Off	7	——	——
5-day B.O.D. in p.p.m.	628	87	52
Per Cent Reduction	——	86	92
Oxygen Consumed in p.p.m.	863	218	175
Per Cent Reduction	——	75	80
Total Organic Nitrogen in p.p.m.	36	16	14
Per Cent Reduction	——	55	61
Free NH_3 p.p.m.	38	1.8	2.3
Nitrite Nitrogen in p.p.m.	——	1.4	1.3
Nitrate Nitrogen in p.p.m.	——	3.0	2.7
Total Solids in p.p.m.	2442	1859	1823
Per Cent Reduction	——	24	25
Total Fixed Solids in p.p.m.	1236	1246	1235
Total Volatile Solids in p.p.m.	1206	613	588
Per Cent Reduction	——	49	51
Soluble Solids in p.p.m.	2061	1661	1659
Suspended Solids in p.p.m.	381	196	164
Per Cent Reduction	——	49	57
Volatile Suspended Solids in p.p.m.	376	164	136
Per Cent Reduction	——	58	64
Average pH	7.1	8.1	8.1
Average Dissolved Oxygen, p.p.m.	——	5.9	5.9
Reducible Sugars	121	0	0
Temp. °C.	21	17	17

* Average of six complete tests.

Winter samples were obtained at this plant when the air temperature at the time of collection was 8° C., the temperatures of the filter influent and effluent being 14° and 5° C., respectively. Analysis of these samples showed a five-day B.O.D. reduction from 1490 parts per million to 270 p.p.m. or 82 per cent. This open filter has been in operation two winters and very little difficulty of any kind has been encountered.

BIOLOGICAL STUDIES

In conjunction with the chemical studies a series of biological observations were made at two of the treatment plants. These observations were made on samples of slime covering the rock at the surface of the

filter, fresh slough-off from the filter, and the sludge in the secondary settling tank. Table IV is a tabulation of the predominating protozoa observed in the samples.

TABLE IV.—*List of Protozoa Found on Milk Waste Treatment Units*

	Class.	
A. Rock at Filter Surface.	Sarcodina	Amoeba (several species)
		Arcella
		Difflugia
	Ciliata	Chilodon
		Colpidium
		Paramecium
		Vorticella
		Opercularia
		Stylonychia
	Mastigophora	Peranema
		Minute Flagellates
B. Fresh Filter Slough-off.	Mastigophora	Peranema
		Minute Flagellates
	Sarcodina	Amoeba
		Arcella
		Difflugia
	Ciliata	Paramecium
		Stylonychia
		Lionotus
		Opercularia
		Chilodon
		Colpidium
		Colpoda
		Aspidisca
		Oxytricha
		Vorticella
C. Sludge Deposits of Secondary Settling Tank.	Mastigophora	Peranema
		Minute Flagellates
	Sarcodina	Arcella
		Difflugia
	Ciliata	Opercularia
		Metophus
		Colpoda

It is interesting to note that the forms found on these milk waste treatment filters are in general the same as those found by James A. Lacey, Research Zoologist of the New Jersey Agricultural Experiment Station, to be the most abundant on municipal sewage filters. The data

obtained by Mr. Lacey indicated that *opercularia*, minute flagellates, *colpidium*, *chilodon*, and *amoeba* are the most common forms found in the top foot of the filter. These forms were very abundant in the top portion of the milk waste treatment filters studied.

The samples of fresh filter slough-off, collected in the channel leading from the filter to the secondary settling tank, contained *aspidisca*, *opercularia*, minute flagellates and *stylonychia* in great abundance. Numerous *difflugia* were also observed, together with *arcella*, *paramecium*, *paranema* and *lionotus*.

The sludge from secondary settling tanks showed decided decrease in amount and diversity of biological life as compared with the fresh slough-off. The forms observed in this material were minute flagellates, a few *peranema*, *difflugia* and a few ciliates. The decrease both in number of protozoa and variety in the secondary settling tank sludge was undoubtedly due to the fact that anaerobic conditions exist in the bottom of the tank and as the majority of protozoa are either aerobic or facultative anaerobes it is natural to expect a rather limited number present in the sludge.

Summary

Studies of milk waste treatment conducted in Wisconsin at an experimental station at De Forest, Wisconsin, during the period 1927 to 1930, have led to the installation of trickling filter systems at various types of milk plants in the state where serious pollution problems existed. On data and experience resulting from the operation of these milk waste treatment systems, are based certain conclusions, which are as follows:

1. Quantities and pollutional strengths of wastes produced by the same type as well as different types of milk plants vary considerably, for example, ranging from 70 to 250 gallons in volume and from 2.7 to 7.5 population equivalent per 1,000 pounds of milk intake at condenseries; necessitating that each waste disposal situation be considered a special problem.

2. Pan water discharged by condenseries, averaging about 1,500 gallons per 1,000 pounds of milk condensed and having an average five-day B.O.D. of about 22 parts per million, may be discharged in many cases directly to a stream without treatment.

3. Properly designed and operated trickling filters, built of crushed rock, 1 to 2 inches in size, 6 to 8 ft. in depth, and having a capacity of 80 cubic feet of rock per pound of five-day B.O.D. applied per 24 hours, will normally reduce the pollutional strength of the wastes (as measured by the five-day oxygen demand test) 80 per cent or more.

4. Rotary distributors, provided with facilities for controlling the filter dosage period, have proved more satisfactory than tipping troughs for applying wastes to the filters. Housing of filters for satisfactory winter operation under Wisconsin conditions has not been found essential.

5. Biological studies showed that the same organisms found on municipal sewage filters developed on the milk waste filters. Predominating forms are listed.

While considerable progress has been made in development of effective treatment for milk wastes, much remains to be accomplished in application of information obtained in the prevention of nuisances and objectionable stream pollution.

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THE POLLUTIONAL CHARACTER OF FLAX RETTING WASTES

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Flax has been grown in Oregon since 1849. Samples of straw and fibre were given first place in direct competition with the best that Ireland and Belgium, long recognized as leaders, had to offer at the Centennial Exposition in 1876 at Philadelphia, Pennsylvania. Trials at the Oregon Agricultural Experiment Station and numerous field plantings through a long period of years have demonstrated that both seed and fibre flax are successful on many Western Oregon soils. The yield and the quality of the product are good.

At the present time about 3,000 tons of flax straw are produced in the Willamette Valley near Salem, Oregon. The entire crop is purchased by the State of Oregon and put through their retting and scutching plant, the largest of its kind in the United States. The cleaned fibre is sold to the spinning mills in Salem or shipped out in bales.

Flax Fibre Production.—The flax plant is a cultivated annual; the stem is commonly solitary, erect with branches only near the top. The

Table I.—*Reduction in B.O.D. of Flax Retting Wastes Due to Chemical Treatment*

Treatment	Chemicals in lbs.	5 Day Basis		20 Day Basis	
		Lb. O_2 Required	% Reduced	Lb. O_2 Required	% Reduced
Raw Waste	—	127	—	235	—
0.5% lime	222	75	41.0	160	31.9
0.5% lime and 0.25% alum	222 111	93	26.8	183	22.1
0.5% lime and 0.1% $FeSO_4$	222 44	82	35.4	168	28.5

flowering tips of the branches bear the seed bolls. The crop is usually pulled when the seed is about two-thirds mature. The flax bundles are shocked, dried, deseeded, and retted.

The flax fibres are bound to the outside of the woody stem by a rather gummy exterior or bark. Retting is the process of loosening the fibre by fermentation of this gummy matter or pectose. It is sometimes done by use of chemicals, but it is thought by many that the best qual-

ity of fibre is secured from flax straw that is retted in water. There are three principal methods of water retting: dew retting, pond or tank retting, and canal or river retting. The factors that appear to make the difference in retting are the quality and temperature of the water, the current, if any, and the bacteria. In the Willamette Valley it is more economical to ret in large tanks and maintain the temperature of the water from 70 to 90° F., thereby reducing the retting time to 5 or 10 days.

In the flax plant, the active retting agent is a microorganism of the bacilli group. In the retting tank, all of the dissolved oxygen disappears in the first twenty-four hours, consequently the process is assumed to be one of anaerobic decomposition. This group of bacteria seems to thrive in a weak acid solution (pH 4.8) and induce decomposition at a maximum rate at about 85 to 90° F. During fermentation the pectose is broken down into pectine and pectic acid. The pectic acid is insoluble in water and remains attached to the fibre until modified or removed by subsequent operations. Pectine is soluble in water and washed from the flax during retting. Since the flax wastes are high in bacterial load and in dissolved matter, a large amount of dissolved oxygen is required to decompose or stabilize them. Consequently, if they are dumped into the stream, the oxygen content of the waters will be materially reduced.

Stream Pollution.—The effect of pollution in any river may be classified into two main divisions; first, that which affects the public health; and second, that which affects stream life. Flax wastes are not likely to affect public health directly, but indirectly the dissolved oxygen demand of the flax waste on the stream may hamper the natural control of pathogenic bacteria. According to the "Sanitary Survey of the Willamette Valley,"[1] the lower portion of the Willamette River is being deoxygenated to a point where it will not support higher forms of stream life.

It is estimated that the flax wastes from the Salem retting plant have a population equivalent of approximately 18,000 people. This indicates the necessity of treatment of the retting wastes if at all practicable. It also points to the fact that an enlargement of the present plant or the establishment of any new plants in the Willamette Valley will require treatment of the wastes before they are dumped into the river.

Salem Retting Plant.—The Salem Flax Retting plant consists of 16 concrete tanks, each 18 ft. by 48 ft. by 6.5 ft. deep, with a total capacity of 200 tons of flax straw, operated on a 10-day cycle. The tanks are filled with water at 85° F. and each day sufficient hot water is added to maintain that temperature. It is estimated that there are 5,300 gallons of retting wastes discharged per ton of flax straw. As indicated

by Figure 1 the waste has an average 20-day biochemical oxygen demand of 4,900 parts per million. It would require 217 pounds of dissolved oxygen to stabilize the 20-day demand of the waste from one ton of straw. Due to the fact that the retting wastes are discharged to the river at a point of heavy industrial pollution, their effect upon the dissolved oxygen content of the river is important from the standpoint of conservation and propagation of fish life. Any extension of the flax industry will aggravate the situation during the low water period which extends throughout the dry summer months, the retting season.

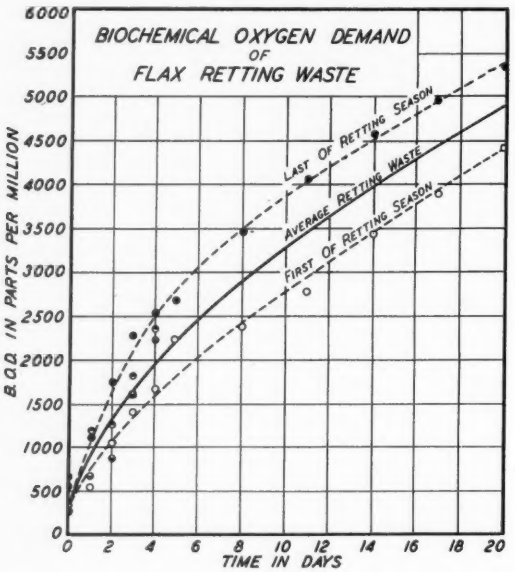

FIG. 1.

Character of Wastes.—Figure 1 shows a marked difference between the B.O.D. at the first of the retting season and toward the last. The reason is probably due to a higher efficiency in the retting process as the season progresses, resulting in the solution of an increasing amount of organic matter. This increase, no doubt, is due to a more effective seeding of the tanks with bacteria and a more favorable air temperature. However, an average concentration of approximately 4,200 p.p.m. of total solids is present in the wastes. The solid material is practically entirely in solution, the small amount of suspended matter being in the form of scattered straw particles. The specific gravity of the wastes is about 1.0075 at 25° C. and the pH value is from 4.82 to 4.88.

The relative stability of the waste, as based upon a 20-day demand, is given in Figure 2. The equation of stability as calculated from this curve is $S = 24\ t^{0.475}$, where S is the per cent stabilized and t the time in days.

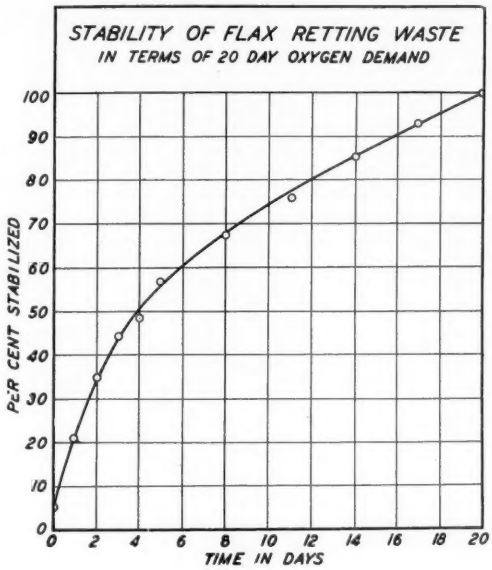

FIG. 2.

Treatment of Wastes.—Figure 3 presents graphically the results of laboratory treatments to reduce the B.O.D. of the waste. A. I. Rossolimo [2] has described treatment with lime and alum. Applying Rossolimo's quantities to the Salem plant, the wastes would require 198 lb. of lime and 99 lb. of alum to one ton of flax straw to produce his asserted reduction. A. J. H. Gauge [3] recommends 2.5 lb. of lime and 2.0 lb. of alum per 100 gal. of waste which amounts to 132 lb. of lime and 106 lb. of alum for the treatment of one ton of straw under the Salem conditions. Both authors report a 50 to 60 per cent reduction in the dissolved organic matter. Chemical treatment was followed by biological purification in both of the above reported tests.

Preliminary tests, using varying quantities of lime alone, indicated 0.5 per cent by weight of solution to be the most effective. This amount represents 222 lb. of lime per ton of straw which is considered to be a close check of the foregoing figures. Table I gives the 5- and 20-day reductions in B.O.D. expressed both as per cent reduction and oxygen re-

quired in pounds per ton of straw. It will be noted that the optimum value of 0.5 per cent lime was used in all trials. No tests were made to determine the reduction of organic matter.

It is considered that chemical precipitation of the flax retting wastes with lime, or lime and some flocculating chemical, when combined with biological filtration, will result in a highly stabilized effluent that may be disposed of by dilution.

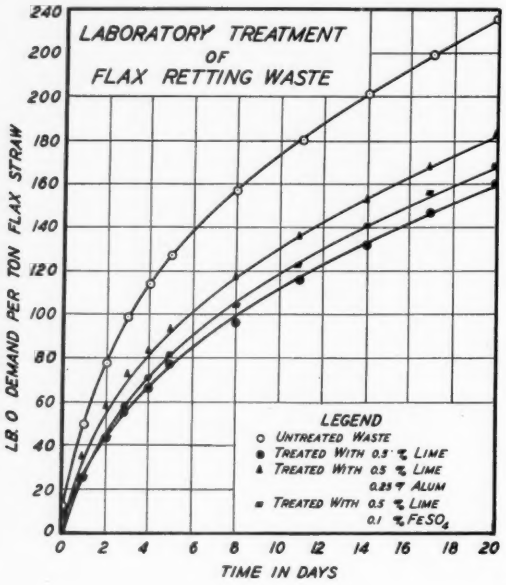
LABORATORY TREATMENT
OF
FLAX RETTING WASTE
LB. O DEMAND PER TON FLAX STRAW
TIME IN DAYS
LEGEND
UNTREATED WASTE
TREATED WITH 0.5 % LIME
TREATED WITH 0.5 % LIME 0.25 % ALUM
TREATED WITH 0.5 % LIME 0.1 % FeSO4

FIG. 3.

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2. A. I. Rossolimo, "Waste Waters from Flax Retting," *Trans. Central Comm. for Protection of Water Reservoirs from Pollution by Industrial Wastes* (Moscow), No. 8, 7–27 (1928). *Chemical Abstracts*, 23, 3766 (1929).
3. A. J. H. Gauge, "Disposal of Flax Retting Effluents," *J. Soc. Chem. Ind.*, 41, 177T (1922).

Editorial

P.W.A. BOARD APPROVES ACTIVATED SLUDGE PROCESS FOR SANITARY DISTRICT OF CHICAGO

RECOMMENDS EXPERIMENTS ON CHEMICAL PRECIPITATION IN WEST SIDE IMHOFF TANKS

A significant report was made April 30, 1934, to Hon. Harold L. Ickes, Administrator of the Federal Emergency Administration of Public Works, by the Board of Engineers appointed by him to review the plans and specifications of the Sanitary District of Chicago for the West-Southwest Side Sewage Treatment Works. The Board was composed of Daniel W. Mead, Chairman, distinguished hydraulic engineer of the University of Wisconsin, with broad experience in consulting work in the U. S. and abroad; W. B. Storey, nationally known railroad executive; Brig. Gen. C. W. Kutz, retired, U. S. A. Corps of Engineers; and Joshua D'Esposito, P.W.A. State Engineer for Illinois, and one of Chicago's most prominent engineers. The board was ably assisted by G. Gale Dixon, Hydraulic and Sanitary Engineer of Youngstown, Ohio; S. W. Freese of the firm of Hawley, Freese and Nichols, Hydraulic and Sanitary Engineers, Fort Worth, Texas; E. F. Robinson, Civil Engineer, Syracuse, N. Y.; and C. W. Post, Electrical Engineer, Chicago.

The Sanitary District of Chicago entered into two contracts with the Federal Government December 16, 1933, whereby the Government agreed to purchase bonds for the financing of the construction work of the District. Out of a total loan and grant of $41,948,000, there was available for construction of the West-Southwest Side Sewage Treatment Works the sum of $21,646,000. For the latter work the contract provides for an engineering survey at the expense of the Borrower by a Commission to be appointed by the Government to review the plans and specifications for the West-Southwest Side Treatment Works, as to general method and details of sewage treatment. The Commission was to make such survey as was necessary to permit the District to take advantage of any change in the development of the art of sewage disposal. Five months were allowed for the survey, that is, until May 16, 1934. The report was completed by the Board April 30, and released by Administrator Ickes May 5, 1934.

The existing West Side Treatment Works of the Sanitary District now comprises two batteries of Imhoff tanks and a third battery under construction, for which $1,314,000 was provided, over and above the $21,646,000. In brief, the Board diverged from the order of construc-

tion laid down by the U. S. Supreme Court, under which the Sanitary District had been proceeding, in that it recommended that the Southwest Side activated sludge plant be built before the West Side Imhoff tanks were supplemented by complete treatment, and it was also recommended that further consideration be given as to the best method of supplementing the Imhoff tank treatment before actual construction.

The general significance of the report, beyond its value to the Sanitary District of Chicago, is due to the continuous, detailed and exhaustive attention devoted to the recently developed chemical processes of sewage treatment. Many reports were furnished to the Board by the Sanitary District comprising the investigations of chemical processes made by the staff of the District. In addition, all possible opportunity was given to the owners of chemical processes to estimate the efficiencies and costs of their processes, as adapted to the problem of the Sanitary District. Further, experts were consulted briefly to state their opinions concerning the suitability of chemical processes for the Sanitary District treatment problem. The report of the Board is therefore of considerable interest and significance with regard to the status of chemical treatment, as well as all other processes of sewage treatment that might be considered by the Sanitary District, including trickling filters, with or without pre-aeration.

The conclusions of the Board, in addition to the recommended change from the West Side to the Southwest Side activated-sludge plant, are that there is an unreasonable waste of water in Chicago, which should be curtailed as rapidly as possible, so that the capacity of future sewage treatment plants may be predicated upon a more reasonable use of water. The recommendations with regard to type of treatment are as follows:

"The plan of the Sanitary District of Chicago for treating the sewage from the Southwest side by the activated sludge process is in accord with the best modern practice."

"Although lower in first cost, the cost of operation, including fixed charges, is such that chemical processes of sewage treatment, as now developed, are not applicable as primary methods of purification of Chicago sewage, but further consideration should be given to their use in determining the best method of supplementing the Imhoff tank treatment."

"The current construction program makes no provision for the disposal of sludge from the North and Southwest Side. Sludge disposal is essential for complete treatment, and must be accomplished prior to 1939."

"The District engineers expect to dispose of the sludge by incineration and experiments on such a process are now under way. An alternative process is available, involving separate digestion and dewatering of

the sludge, at a cost approximating that which has been estimated for incineration."

"The best method of sludge disposal should be determined as soon as practicable, but the large investment required for a permanent plant should be predicated only upon dependable information as to the merits of incineration, which is still lacking."

"Accordingly, the Board does not include any expenditure for sludge disposal in its modified construction program."

The report finds, however, that "disposal by incineration, if it can be accomplished at reasonable cost, is highly desirable."

It also states that high construction standards prevail in Sanitary District works already built.

The body of the report contains detailed estimates of costs and further consideration of the possibilities of various types of treatment. A brief description of the activated-sludge process ends with the statement that "a well operated activated-sludge plant will remove from 88 to 93 per cent of the suspended solids and from 90 to 95 per cent of the 5-day biochemical oxygen demand."

With regard to chemical treatment the report states that "chemical precipitation effects a larger removal of suspended solids than plain sedimentation, depending upon the amount, character, and method of application of the chemicals used. It does not, however, affect the organic impurities held in solution as do the trickling filter and activated-sludge processes. The amount of sludge is from 2 to 3 times (dry weight basis) that of the suspended solids removed from the sewage, due to the precipitation of inorganic solids. Chemically precipitated sludge digests more slowly than does sludge from plain sedimentation and may produce objectionable odors."

"In order to obtain a more satisfactory effluent, chemical precipitation has been supplemented by filtration in several experimental plants. Sand, anthracite coal, magnetite and zeolite have been tried as filtering or straining media. In the Laughlin process a magnetite filter is agitated by a solenoid operated magnet while the filter is being backwashed. The Guggenheim process utilizes a zeolite filter which is regenerated with brine."

"The best results that can be obtained from chemical precipitation are such that some supplemental treatment must be provided to produce the degree of treatment required for the Southwest Side Treatment Works. The accompanying estimate of cost provides for zeolite filtration (Guggenheim process) the experimental data indicating that with the chemical usage assumed, this form of filtration will provide a degree of purification equal to that obtained by the activated sludge process."

"As between the activated sludge process and chemical processes, the cost comparisons clearly support the use of the activated sludge process for the treatment of the Southwest Side sewage."

"It is not equally clear to the Board that the activated sludge process is the best method of supplementing the treatment in the West Side Imhoff tanks. . . . In the opinion of the Board, tests and experiments heretofore made should be supplemented by large scale tests at the West Side plant to determine whether some economical combination of Imhoff tank treatment and chemical treatment, either with or without filtration, can be found which will so improve the quality of the effluent as to make further treatment unnecessary."

Estimates of cost as made by the Board for the Southwest Side Works, with an estimated flow of 400 m.g.d. and comparisons of activated sludge treatment and chemical treatment, followed by zeolite filtration, were as follows:

TABLE I.—*Cost Estimates—400 M.G.D. Southwest Treatment Plant*

		Activated Sludge	Chemical Precipitation-Filtration
Treatment			
	Original Cost, per M.G.D.	*$26,600* [1]	*$18,800* [1]
Per Mil. Gal.	Fixed Charges	4.30	3.05
	Operation	4.22 [2]	10.60
	Sub-Total	8.52	13.65
Sludge Disposal			
	Original Cost per M.G.D.	*$ 5,400*	*$ 6,000*
Per Mil. Gal.	Fixed Charges	1.01	1.12
	Operation	3.63	3.59
	Sub-Total	4.64	4.71
Total			
	Original Cost per M.G.D.	*$32,000*	*$24,800*
Per Mil. Gal.	Fixed Charges	5.31	4.17
	Operation	7.85	14.19
	Total	13.16	18.36

[1] Does not include royalty.

[2] Includes power cost based upon .5 cu. ft. of air per gallon of sewage.

All cost estimates are based upon sludge disposal by mechanical dewatering and incineration.

In the summary, the Board report states:

"The Board has given considerable time and attention to various chemical processes of sewage disposal. Plants of this character are attractive on account of the greater compactness and lower cost of construction. However, the high cost of chemicals and, in some cases, the

relatively inferior quality of the resulting effluent precludes this process from consideration as a primary method of disposal under conditions in Chicago, although its consideration as a supplementary method either for the treatment of the Imhoff tank effluent or for final treatment of the effluent of other processes, merits further consideration."

During the course of the investigations, four expert consultants were called by the Board to give their written answers to four questions relating to the choice of processes, particularly referring to chemical treatment. These four men in the order of their appearance were Prof. John H. Gregory, George W. Fuller, Harrison P. Eddy, and Prof. A. M. Buswell. The question of most general interest was, "Do chemical processes of sewage treatment merit serious consideration as a primary means of treating Chicago sewage?" The replies were as follows:

Professor Gregory: "Chemical precipitation as a primary means of treating Chicago does not, in my opinion, merit serious consideration."

Mr. Fuller: "Chemical precipitation processes merit serious consideration as a primary step only in relation to the sewage passed through the Imhoff tanks at the West Side when the effluent of these tanks may be sent direct to the channel or to a secondary treatment plant in which sprinkling filters are included."

Mr. Eddy: "By a 'primary means' I understand you to mean the principal means of treatment upon which the District would rely chiefly for the degree of purification to be secured. Chemical treatment does not afford at equal or less cost any improvement in the quality of effluent over that which can be secured by the activated-sludge process planned by the District."

Professor Buswell: "Chemical precipitation is much more reliable, and at the same time, more flexible than either of the aerobic biological purification processes (activated sludge and trickling filters) that it is recommended for consideration. The biological processes are subject to serious interruptions and while they may be capable of a higher percentage purification than that obtained as yet by chemical precipitation, they cannot be depended upon for a uniform effluent."

This very interesting report is to be published by the Sanitary District of Chicago. The report states that throughout its consideration of the problem, it had the full cooperation of the Chief Engineer, Philip Harrington, and that the staff of the Sanitary District worked in close cooperation with the special assistants of the Board. It is expected that the design of the Southwest Side Works will be pushed as rapidly as possible and that consideration will be given simultaneously to the experimental work recommended by the Board.

F. W. MOHLMAN, *Editor.*

Proceedings of Local Associations

A SUMMARY OF THE TENTH ANNUAL CONFERENCE OF THE MICHIGAN SEWAGE WORKS ASSOCIATION

The Tenth Annual Conference of the Michigan Sewage Works Association was held at Michigan State College, East Lansing, Michigan, March 26–30, 1934. The first three days were devoted to a short course in laboratory practice in sewage analysis and plant hydraulics followed by a two-day program of papers and discussions. About 70 persons were registered during the conference and 25 attended the three-day short course. The laboratory work was given by Professors Eldridge, Mallmann and Theroux of Michigan State College faculty. The schedule of courses presented is given below.

Prof. C. L. Allen, Professor of Civil Engineering, Michigan State College, welcomed the conference to the College. Mr. J. A. Volk gave an interesting description of the Traverse City sewage treatment works which have now completed a full year's operation. A notable achievement has been the reduction of the pollution of Little Traverse Bay as indicated by the results of frequent bacteriological surveys.

Mr. James R. Rumsey gave a short talk on the experience at the Grand Rapids plant with small gas engines, using sewage gas for fuel. Mr. Rumsey had analyzed the performance of the engine very carefully, accounting for all of the hours which the engine was out of service, allocating the hours to the various reasons for shutting down. In general, it may be said that the engine gave very satisfactory service.

A brief talk by Deputy Attorney General Gerald K. O'Brien on the legal phases of sewage treatment was well received. Mr. O'Brien stressed the manner in which legal decisions had followed economic conditions.

A paper by Mr. N. G. Damoose showed the effect of and results being obtained on the Grand River by the sewage treatment plant at Grand Rapids. A complete series of graphs showed the variation of D.O., B.O.D. and bacteria at several sampling stations above and below the treatment plant. These data were compared with a series of determinations made before the plant was constructed and indicate the degree of improvement brought about by the plant.

The annual "Question Box" provided the usual quota of ardent discussion. Time available limited the number of subjects discussed.

Mr. Thomas Doyle of Pontiac gave an interesting talk on the experiences at Pontiac from the plant operator's viewpoint. Particularly interesting was the comparison brought out of the hit or miss methods used

in operating a plant during his first years of operation as compared to the scientific control of later years.

Dr. W. L. Mallmann of the Michigan State College Bacteriology Department gave an interesting discussion of some data obtained in chlorinating sewage at Flint and Traverse City. The use of a sodium thiosulphate sampling bottle would apparently give a more nearly correct picture of the effectiveness of chlorination.

Mr. George Hubbell of Hubbell, Hartgering & Roth discussed the effect of inlet arrangements on sedimentation tanks. He brought out an interesting comparison of various types of inlets on a model tank constructed in accordance with the old Dearborn Imhoff tank plans. As a result of the studies of the model a new type of inlet was developed and installed with a marked increase in sedimentation efficiency resulting.

An instructive illustrated talk on the use of small activated sludge plants with mechanical aerators was given by Mr. J. E. Holst of the Simplex Aerator Corp. of Chicago. The adaptability of this type of installation was admirably shown.

The conference membership was very happy again to have Mr. Walter Sperry of Aurora, Illinois, present. Mr. Sperry gave one of his interesting talks on human relationship in connection with the routine plant operation. In addition he talked about certain phases of plant operation and discussed quite fully the development and use of a sludge metering device which has been quite helpful to him at Aurora.

At the Annual business meeting Mr. Pierre Welch of Flint was elected *President* for the year 1934; Mr. J. A. Volk of Traverse City, *Vice-President;* Mr. E. C. Miller of Dearborn, *Director;* and Mr. W. F. Shephard, *Secretary-Treasurer.*

The conference voted to endorse the adoption of rules and regulations for certification of plant operators by the Michigan Department of Health.

W. F. SHEPHARD,
Secretary-Treasurer

MICHIGAN STATE COLLEGE

SCHEDULE OF COURSES—THIRD ANNUAL SHORT COURSE

For Michigan Sewage Works Operators

Date & Time	Groups			
	Elementary		Advanced	
	A	B	C	D
Monday, March 26				
9:00 A.M.	Registration	Registration	Registration	Registration
9:30–10:30	Lecture	Lecture	Lecture	Lecture
10:30–12:00	Total Solids	Ammonia N	Standard Acid	Fertilizer Anal., P_2O_5
1:00–3:00 P.M.	Suspended Solids	Organic N	Advanced Bact.	Advanced Bact.
3:00–5:00	Bacteriology	Bacteriology	Standard Acid	Fertilizer Anal., P_2O_5
7:00–9:00	Plant Hydraulics	Plant Hydraulics	Plant Hydraulics	Plant Hydraulics
Tuesday, March 27				
8:00–12:00 A.M.	Settleable Solids, Sludge Moisture and Vol.	Organic N Nitrite N	Standard Alkali	Fertilizer Anal., Nitrogen
1:00–3:00 P.M.	Dissolved Oxygen B.O.D.	Nitrate N	Advanced Bact.	Advanced Bact.
3:00–5:00	Bacteriology	Bacteriology	Std. Permanganate Std. Oxalate	Fertilizer Anal., Humus
7:00–9:00	Plant Hydraulics	Plant Hydraulics	Plant Hydraulics	Plant Hydraulics
Wednesday, March 28				
8:00–12:00 A.M.	B.O.D. Oxygen consumed	Residual Chlorine Cl. Demand	Std. Thiosulphate	Chlorides Fats
1:00–3:00 P.M.	Completion of Tests	Completion of Tests	Advanced Bact.	Advanced Bact.
3:00–5:00	Bacteriology	Bacteriology	Alkalinity	Hydrogen Sulphide

Note: Determinations as above listed were part lecture and part laboratory.

Fuller and McClintock announce that on April 30 they moved their Engineering Offices from 170 Broadway, where they were located for nearly 30 years, to larger and specially planned quarters on the 16th floor of the Park-Murray Building, 11 Park Place, New York City.

NOTICE OF EIGHTH ANNUAL CONFERENCE, PENNSYLVANIA SEWAGE WORKS ASSOCIATION, STATE COLLEGE, PA.

June 27, 28, 29, 1934

The Eighth Annual Conference of the Pennsylvania Sewage Works Association will be held June 27 to 29, 1934, at State College. The following program has been arranged:

WEDNESDAY, JUNE 27, 1934

Afternoon Session

Eastern Standard Time

1:00 Registration Opens.

8:00 "Some of the Newer Developments Abroad," Dr. W. Rudolfs, Chief, Department of Water and Sewage Research, New Jersey Agricultural Experimental Station, New Brunswick, New Jersey.

THURSDAY, JUNE 28, 1934

Morning Session

8:30 Registration.

9:30 Address of Welcome, Professor F. G. Hechler, Assistant Director of the Engineering Experiment Station, Pennsylvania State College, State College, Pennsylvania.

9:45 "The Trickling Filter Fly: Its Habits and Control," Gordon M. Fair, Associate Professor of Sanitary Engineering, Harvard University, Cambridge, Mass.

Discussion, Harry J. Krum, City Chemist, Allentown, Pa.

11:00 "Elutriation and Filtration of Sewage Sludges," Albert L. Genter, Chemical and Mechanical Engineer, Baltimore, Md.

Discussion, C. E. Keefer, Principal Assistant Engineer, Bureau of Sewers, Baltimore, Md.

12:00 Recess.

Afternoon Session

1:15 "Comparison of Vacuum Filters and Open and Glass Covered Sludge Drying Beds," Frank Woodbury Jones, George B. Gascoigne, Consulting Engineer, Cleveland, Ohio.

Discussion, Wellington Donaldson, Fuller & McClintock, Engineers, New York, N. Y.

2:15 Business Session.
Special Report of Operators' Licensing Committee.

3:00 Sewage Treatment Plant Operators' Symposium.

1. "Hershey Sewage Treatment Works and its Operation," H. N. Herr, Chief Civil Engineer, Hershey Estates, Hershey, Pa.

Discussion, I. M. Glace, District Engineer, Pennsylvania Department of Health, Harrisburg, Pa.

2. "The Operation of the Hanover Sewage Treatment Plant and its Sewage Ejector Stations," C. A. Eckbert, Borough Manager, Hanover, Pa.

Discussion, R. C. Gorman, Jr., District Engineer, Pennsylvania Department of Health, Harrisburg, Pa.

4:30 Recess.

7:00 Dinner and Round Table Discussion, H. E. Moses, Leader, Assistant Chief Engineer, Pennsylvania Department of Health, Harrisburg, Pa.

FRIDAY, JUNE 29, 1934

Morning Session

9:00 "Plant Operation," W. L. Stevenson, Chief Engineer, Pennsylvania Department of Health, Harrisburg, Pa.

Discussion, H. M. Beaumont, Assistant Engineer, Bureau of Engineering, Philadelphia, Pa.

10:00 "Notes on Sedimentation," L. V. Carpenter, Head, Department of Civil Engineering, University of West Virginia, Morgantown, W. Va.

Discussion, F. B. Foote, Morris Knowles, Inc., Engineers, Pittsburgh, Pa.

11:00 "Sludge Digestion at Meadville," C. H. Young, District Engineer, Pennsylvania Department of Health, Meadville, Pa., and Roy L. Phillips, City Engineer, Meadville, Pa.

Discussion, J. R. Hoffert, Civil Engineer, Pennsylvania Department of Health, Harrisburg, Pa.

12:00 Adjournment.

L. D. MATTER, *Sec'y-Treas.*

Book Reviews

A SPANISH TREATISE ON SEWERS AND SEWAGE DISPOSAL

Book Review. Municipal Sanitation and Sewage Purification. (Saneamiento de Poblaciones y Depuracion de Aguas Residuarias.) By Jose Luis Escario. 303 pages, 165 illustrations. Published by Talleres Graficos Herrera, Hermosilla 44, Madrid, Spain. Price 60 pesetas ($8.25). This book was published by the author following an extensive trip throughout the United States, England and Germany. As a consequence the subject matter is largely concerned with the present status of sewage treatment in these countries, while a small part of the text refers to Spanish sewer systems and to the almost negligible sewage treatment works in Spain. The remarkable feature of the book is that the American and English data are so accurate and well-chosen that the book should be of real interest for American or English readers. The Castilian text can be understood fairly well by anyone with a knowledge of French, and the cuts and charts are universally intelligible. There are 54 full page half-tone plates.

The discrimination of the author, in selecting the most modern and most interesting plants of various types in the three countries visited by him, is to be commended. The book can be highly recommended to South American technicians, as a reliable presentation of the state of the science of sewage treatment in the United States. The discussion of sewage analyses, per capita quantities, D. B. O. (demanda bioquimica de oxigeno) and sludge digestion (including thermophilic) are quite interesting, reliable and up-to-date.

The author intends to furnish owners of the book with appendices to be issued annually, which will include the most important developments in sewage treatment during the year. This is a novel and unique feature. The address of the author is Calle Duque de Liria, No. 6, Madrid, Spain.

F. W. Mohlman

GAS HAZARDS IN SEWERS
FLOW CHARACTERISTICS OF SEWAGE SLUDGE

Monographs from *Gesundheits-Ingenieur*

Series II, Volume 13. Gas Hazards in Sewers (Die Gefährdung der Kanalisationsanlagen durch Gase). By Dr. Ing. Kurt Ripperger. R.

Oldenbourg, Munich, 1933. Price, 5.2 marks. ($2.10.) This monograph of 28 pages, each 8½ by 12½ inches, is a thorough presentation of the history of sewer explosions in 26 German cities, extending back to 1889. In 40 years there were 47 deaths in these cities from sewer explosions or fires; there were 87 records of explosions or gas poisonings. In 51 cases benzine (gasoline) was responsible for the accident. Brief reference is given to similar accidents in a few of the larger European cities.

The properties and physiological effects of the various gases are described as well as methods for their detection and analysis. Especial attention is given to the significance and danger of gases from garages. Benzine traps of various types are described. Artificial aeration of suspected sewers is recommended, and safety rules are stressed. A bibliography of 44 references is appended.

Series II, Volume 13. The Flow Characteristics of Sewage Sludge (Die Fliesseigenschaften von Abwasserschlamm). By Dr. Ing. Wilhelm Merkel. R. Oldenbourg, Munich, 1934. Price 4.5 marks ($1.80). This monograph contains 28 pages. It includes a discussion of the properties of sewage sludge, particularly as they affect its pumpage. Reference to experimental tests at Chicago, Baltimore and Syracuse is illustrated by curves showing loss of head in the Chicago experiments. The investigations of the author are then described. The tests were made at Stuttgart and Nuremburg, using conduits 20 cm. in diameter. The field tests were correlated with laboratory tests of viscosity, devised particularly for this study. The moisture contents of the sludges investigated were comparatively low, ranging from 84 to 93 per cent, with very few records of sludge with more than 90 per cent moisture. The report therefore does not apply to activated sludge, although the data can be roughly interpolated for sludges with such high moisture contents.

This monograph is of considerable value and should be in the hands of all American engineers who design or operate sewage treatment works.

F. W. Mohlman

Reviews and Abstracts

H. W. STREETER

BIOLOGICAL PURIFICATION OF SEWAGE IN SPRINKLING FILTERS

By H. Blunk

Gesundh. Ing. **56**, 425, 440 (1933)

It is commonly thought that sewage is discharged from a sprinkling filter within 5 or 10 minutes after it is added. However, when a waste containing a large amount of salt from a dye works was added intermittently to the domestic sewage of the Soest treatment plant it was found by analysis that the detention period was more than 10 minutes. It appears that certain quantities of salt may accumulate and be discharged gradually. In a 24-hour study of a large sprinkling filter the same amount of salt was found in the influent and effluent, that is, 595.5 kg. in the influent and 585 kg. in the effluent. The small difference is within the limits of error.

In order to investigate the process further an experimental filter 4 m. deep, which had been in operation several months, was treated with 4 cu. m. of sewage per cubic meter of filter material. The sewage was distributed uniformly over a period of 18 hours and measurements were made of the effluent. Five minutes after the loading began a small amount of effluent appeared, increasing up to 20 minutes and from then on, remaining about the same as the influent. After 18 hours the raw sewage flow was stopped, the effluent soon decreased and after about 6 hours stopped entirely. Except for the amount which is retained in the filter, the sewage seems to flow through the filter in 20 minutes. The amount of sewage retained depends upon the surface of the material and the size of the stones used. The smaller the stone the larger the surface available for the purification of sewage. A sprinkling filter of 1 cu. m. content consisting of round material, 2 cm. in size, with irregular surface has a surface of about 200 sq. m. whereas a similar filter with 9 cm. material has a surface of only 40 sq. m. With a biological film only 1 mm. thick there are 200 liters of film in the first filter and 40 liters in the second. As this biological film often consists of about 96 per cent water the amount of water retained in such a filter can be estimated. For example, if 2 cu. m. of sewage per day are added to a sprinkling filter consisting of 1 cu. m. of 2 cm. material the water retained in the biological film can be renewed about ten times per day,

that is, the water remains in the filter about 2.4 hours. The water falling from above displaces the water in the film below. To be sure, there is the same amount of liquid in the effluent as in the influent but the sewage has had the opportunity of remaining from 0.5 to 5 hours in the filter, during which time it has been purified by the micro-organisms. If the detention period of a sprinkling filter is not affected by the biological film then this period should be the same for filters of the same height and kind. It was found, however, by analyses of salt content, that doubling the load halved the detention period. If a sprinkling filter is overloaded the liquid in the biological film is replaced and the remaining sewage passes through the filter unpurified.

As the biological film has a moisture content of about 96 per cent, one liter of film would contain 960 c.c. of water and 40 c.c. of dry material. If one fourth of these dry solids are micro-organisms their volume would amount to 10 c.c. With a moisture content of 90 per cent there would be 100 c.c. of organisms or 110 million organisms in each liter of biological film. Although the oxygen required by the micro-organisms varies under different conditions 1 liter of biological film would require about 136 c.c. of oxygen per hour, which must be obtained from the oxygen dissolved in the water. In a sprinkling filter of 1 cu. m. content there are about 200 liters of biological film. The amount of oxygen required would be $.136 \times 200 = 27.2$ liters of oxygen per hour, or 653 liters per day. If a sprinkling filter of 1 cu. m. capacity can purify 4 to 5 cu. m. of sewage then 1 cu. m. of sewage would require $653/4.5 = 145$ liters of oxygen. Since only part of the oxygen in the air is dissolved in the sewage it is estimated that 30 cu. m. of air per day must be distributed through the sprinkling filter.

With ordinary concentrations, micro-organisms will discharge water equivalent to about 100 times their own volume. If 200 liters of biological film are present in 1 cu. m. of filter and of this 200 liters, 20 liters are organisms then 20×100 or 2 cu. m. of sewage can flow through the bodies of the organisms each day.

The salt content has an important effect on the organisms. For instance, with a concentration of 0.025 per cent the amount of sewage treated per day decreases by one half. With a concentration of 0.5 per cent, the sewage treated is reduced to about one third.

It is estimated that with 200 liters of biological film 5.25 kg. of organic matter per day would be destroyed.

The effect of heat in the biological purification of sewage has not been sufficiently considered. It has been found that at from 5° to 10° C. small animals divide themselves every 23 to 26 hours, from 20° to 25° C. every 11 to 14 hours and at 30° C. every 6 hours. It is just as important to remove the CO_2 resulting as a decomposition product, as

quickly as possible, as it is to supply the large amount of oxygen necessary. Consequently, it is customary in Germany to select large slags for the construction of sprinkling filters. The depth of the sprinkling filter is very important. Greater depth in a sprinkling filter has the advantage of lower construction cost because of smaller ground and foundation area, smaller distributing machine and easier and more equal distribution of sewage on the surface. In Germany sprinkling filters 10 to 13 feet deep have given satisfactory results. With fine material in the filter the supply of oxygen and the removal of carbon dioxide can be accomplished by aeration. A filter 13 feet deep could be aerated with a vacuum of about 1.5 inches so that the power required is negligible.

In order to prove in part the value of these theoretical determinations some results from practice were considered. The sprinkling filter plant in Soest is built of scoriaceous lava and it is estimated that it contains 195 cu. m. of biological film. According to the chemical analysis, 0.284 kg. of organic matter is decomposed per cu. m. of sewage. Since 3500 cu. m. of sewage are purified per day the total organic matter destroyed amounts to 994 kg. or 5.1 kg. per cu. m. of biological film. In Holzwickede, where 600 cu. m. of sewage are treated daily and 81 kg. of organic matter destroyed, the sprinkling filter contains 18 cu. m. of biological film and 4.5 kg. of organic matter are consumed per cubic meter of biological film per day. At Erfurt, the activated sludge plant, which has a capacity of 4325 cu. m., treats about 8100 cu. m. of sewage per day and, according to analysis, 2456 kg. of organic matter are destroyed daily. Since there are 490 cu. m. of activated sludge, the organic matter destroyed per cubic meter of activated sludge per day is about 5 kg. From these three entirely different examples it may be seen that the values agree approximately with the 5.25 kg. determined theoretically. The relatively small difference could be explained by differences in temperature.

From these considerations, calculations for a sprinkling filter would proceed as follows: The analysis of sewage to be treated shows, for example, 500 p.p.m. loss on ignition. A decrease of 75 per cent of organic matter should be obtained so that 375 p.p.m. should be destroyed. Since, of the 1000 cu. m. of sewage treated, 375 kg. of organic matter must be destroyed and since 5 kg. of organic matter would be destroyed by 1 cu. m. of biological film, therefore, 375/5 or 75 cu. m. of biological film would be required. The average annual temperature is only 10° C. so that the amount of filtering material required must be equal to 1.4 (from curve) $\times$ 75 = 85.5 cu. m. With 4 cm. stone there would be 0.1 m. of biological film (from curve) in 1 cu. m. sprinkling filter. The required size is, therefore, 85.5/0.1 = 855 cu. m. of sprinkling filter

(4 cm. size). With 6 cm. size stone $85.5/0.07 = 1220$ cu. m. and with 8 cm. stone $85.5/0.05 = 1710$ cu. m.

G. P. EDWARDS

THE HARMFUL EFFECTS OF SEWAGE IN RIVERS

BY DR.-ING. MAHR, Wuppertal

Gesundh. Ing. **56**, 498 (1933)

In order to study the permissible sewage load on streams a number of German cities have been grouped according to the type of sewage treatment and according to whether the sewage is discharged into a flowing stream or still water. Group I contains the cities whose sewage is either unpurified or only screened, an arrangement possible only with relatively high dilution. Cologne, for example, discharges practically unpurified sewage from 500,000 and 200,000 people and from numerous factories into the Rhine, with a dilution at low water of 1:400. Since the sewage flows along the river banks for some time, the dilution is really less than that, but the pollution can hardly be detected farther than 6 miles down stream. Dusseldorf and Duisburg, where the points of discharge are farther out in the stream, have a greater dilution. The screened sewage of Dresden is so discharged into the river (dilution 1:80 to 1:250) that it is soon distributed over its entire width. Under normal conditions Göttingen has a dilution of 1:50 and during low water a somewhat lower value. The sewage, which is passed through bar screens, is well distributed over the entire river but traces of it may be seen some distance down stream. Kassel, with a dilution of 1:500, has permission to abandon its settling tanks. Under average conditions a dilution of 1:200 seems satisfactory for flowing streams if the coarse material is removed by bar screens and the demands on the stream just below the outfall are small.

Group II contains the cities whose sewage is settled and then discharged into flowing water. Many cities of medium or large size have a dilution of only 1:20 at low water, which seems to be a minimum. Hannover now has a dilution of 1:15 and conditions are not satisfactory. The higher dilution of 1:130 does not satisfactorily handle the settled sewage at Halle where the Saale is already overloaded. At Munich, the sewage, diluted to about 1:10 in a canal, is not well settled but is further purified in fish ponds. Under favorable conditions a well settled sewage requires a dilution of at least 1:20.

Group III includes the cities whose sewage is biologically purified before its discharge into a flowing stream. Conditions are not entirely satisfactory at Gotha and Aachen, where there is a dilution of 1:2 in

low water. Gosslar reports that their sprinkling filters are too small for a dilution of 1:3. With a dilution of 1:10 Essen-Rellinghausen has a well operated activated sludge plant as the river must be well protected for water works and bathing beaches. In general, it seems that ordinary biological purification by sprinkling filter is sufficient if a dilution of at least 1:3 at low water is provided. With a specially good biological treatment a lower value is satisfactory but allowance must be made for rivers which receive a large amount of pollution during storms.

The average efficiency for different types of treatments and the dilutions required are shown in the following table:

Purification Process	Decrease in B.O.D.	Required Dilution	Correction Factor	Corrected Dilution
1	2	3	4	5
Fine Screen	10–15	1 : 200	1.11	1 : 222
Settling Plants	30–40	1 : 20	1.43	1 : 28.6
Sprinkling Filters	60–70	1 : 3	2.50	1 : 7.5
Well Operated Activated Sludge	80–90	1 : 0.5	5.00	1 : 2.5

The values in this table can only be taken as rough averages and must be accurately determined by investigation for use in individual cases. The required dilution takes into consideration only the amount of sewage introduced and not the pollution already present. It is evident that a river with poor clarifying action needs more water than one with good clarifying action in order to dilute the large amount of organic matter to a permissible degree. In Column 3 the values should be recalculated so that the river receives the same amount of organic matter (measured by the B.O.D.) from the sewage purified by different methods. In order to arrive at a comparable value of 100 per cent organic matter, the amount of screened sewage (10 per cent purification) must be multiplied by $100/90 = 1.11$; and settled sewage (30 per cent purification) by $100/70 = 1.43$ (Column 4). Accordingly the larger amount of sewage requires a larger dilution (Column 5). These figures serve as comparative values. For example, with the same amount of pollution, 30 per cent purification requires 28.6/7.5 or about four times as much dilution as does 60 per cent purification and the material in the first case is four times as harmful as in the second. The well-known fact that sludge and coarse organic material load a river much more than the same amount of fine or dissolved organic matter is confirmed in a striking manner.

A river is loaded less by well digested organic matter than by an equal amount of half digested material. This is important if the question of partial purification is considered. It has been generally assumed that the condition of a river would be the same whether the entire amount of sewage was purified in a mediocre manner or a part especially well purified and the remainder correspondingly less. These results seem to indicate that it is advisable to purify the entire amount uniformly.

Group IV includes the cities whose sewage is discharged into stationary or nearly stationary water. In most of these cities solids are removed and the sewage is discharged at some distance from the shore (Kolberg, 492 feet; Königsberg, 1300 feet).

It is difficult to determine the surface necessary for purification in the case of cities where the sewage is discharged into large natural lakes. It seems to be surprisingly small in the case of Rostock.

The cities with sewage fish ponds belong in this same group as they also discharge a settled sewage into practically quiescent water. In Munich and Grafenhainichen the fish ponds treat the sewage from 1050 people per acre. This is also true in Memmingen if the industrial waste is converted to a domestic sewage basis. Although the dilution in small plants is quite large, in Munich it is only 1:3 to 1:5.

Well settled sewage in flowing streams requires a dilution of at least 1:20 whereas in fish ponds the dilution of 1:3 or 1:5 is sufficient. The difference lies partly in the fact that the river receives a larger load from occasional disturbances or rain and partly because the river below the point of discharge contains an unchanged mixture of sewage and pure water. The dilution in a river must be large enough so that fish can live at the sewer outfall. In a pond, although a small area may be strongly polluted, a short distance away there may be large pure areas into which the fish can easily move.

G. P. EDWARDS

SEWERAGE AND SEWAGE DISPOSAL IN THE URBAN DISTRICT OF EASTLEIGH *

BY FREDERICK G. LEE

This paper "is of particular interest for the reason that it describes in detail the process of sludge digestion, gas production and utilization adopted at Eastleigh, where the pumping of the sewage and the heating of the tanks containing sludge is effected by means of the sludge gas."

* Presented at a Southern District Meeting of the Institution of Municipal and County Engineers, March 1, 1934, and reviewed in *The Surveyor*, **85**, 303–306 (March 9, 1934).

Sewerage.—The main collecting sewers provide for a population of 40,000. Flows were calculated at 25 gallons per capita per day with a maximum of 2 gallons in one hour, a maximum rainfall of ½ inch per hour less 32 per cent evaporation and 100 sq. ft. per head of impervious area for houses connected to the sewers or an average of 500 sq. ft. per house. All of the sewage must be pumped, some sewers having been laid 32 feet deep.

Sewage Treatment Works.—The works provide for a population of 27,000 or 675,000 g.p.d. dry weather flow; the surplus is by-passed to storm water tanks, and thence to a land area before discharge to the River Itchen.

The sewage first flows through screens having an effective area of 30 sq. ft. which may be operated either in series or in parallel, and then over a recording weir into a channel delivering either to the storm tanks or the disposal works proper. The dry weather flow next passes through grit channels with back fall inverts at a velocity of 1 ft. per second and then enters an oil and grease interceptor. In this tank, rectangular in plan and fitted with specially placed baffles, the sewage is agitated with air admitted through diffuser plates in the bottom which causes the floating oil and grease to drift through slat barriers to side compartments from where it is removed by hand scoops.

The sewage next passes to the four, two-story sedimentation tanks each with upper and lower compartments of 62,500 gallons and 21,000 gallons capacity respectively. At the apex of the sloping roof of the lower chamber is a concrete shaft terminating in a gas collector. Sludge is removed from these tanks through sloping outlet pipes under hydrostatic pressure to the sludge digestion tanks. The effluent from the sedimentation tanks is treated on twelve 84 ft. diameter circular filters, 6 ft. deep, constructed of coke and gravel, at the rate of 50 gallons per cu. yd. of filter volume. The filter effluent is next conducted to hopper bottom humus tanks 62 by 25 ft. in plan and thence through a series of tumbling bays to the outfall sewer.

Sludge Digestion.—Sludge from the lower compartment of the sedimentation tanks is passed to the four sludge digestion tanks each with a capacity of 92,700 gallons, which are always kept filled. Gas collectors, provided with hand operated scum stirrers, draw off the gas generated during digestion. To facilitate digestion, coils around the inside of the tanks are supplied with water heated by the exhaust waste gases and circulating water of the oil engines in the generating station. The digestion tanks are operated in pairs as primary and secondary units, sludge being pumped from one to the other. Valves at different levels permit withdrawal of the supernatant water, which is returned to the tanks for further treatment. The digested sludge is discharged by

gravity and pumped to underdrained sludge drying beds, 4,000 sq. yds. in area. The liquid is returned to the sedimentation tanks and the dried sludge is sold to farmers.

Operation.—Detailed procedures for operation of the sludge digestion tanks are given, many of which have been more or less experimental during the year the plant has been in service. After sufficient gas had been accumulated one of the 75 h.p. oil engines in the power plant was converted to operate on the sewage gas and works 50 per cent of the time on this fuel. The digesting sludge has been kept at a temperature of 70°–75° F. and the circulating water at 100° F. The gas generated, the quantity of which varies considerably, has an average calorific value of 715.5 B.T.U. per cubic foot.

Costs.—The cost of the sewerage and disposal works was £101,000. A saving of £1,500 per annum has been effected on the maintenance by the new plant, although the population served has increased.

J. K. Hoskins

ADVANTAGES OF THE BIO-AERATION SYSTEM OF SEWAGE PURIFICATION *

By S. Thornhill

In this paper the author enumerates various advantages of the bio-aeration system, raises the question as to what holds back the wider adoption of this method of sewage treatment and in turn demolishes the possible objections to the process that have been advanced.

Advantages.—The principal advantages of the bio-aeration treatment process as stated by the author are freedom from smell, absence of fly production, less fall needed in flow through the plant, smaller area of land required and lower construction cost. The author's experience at the bio-aeration plant at Chesterfield and elsewhere has been that no fly nuisance has ever resulted either on the works or to residents near the works. Areas that could not be used for filter schemes without pumping can be adopted for bio-aeration plants and filters moreover require about one-third more land. Construction costs for percolating filters are approximately one third greater than for a bio-aeration plant.

Comparison of Control Requirements.—The activated sludge process, of whatever type, does require more scientific control but this is a step in the right direction because any system of sewage purification can be rendered totally ineffective by inefficient control. The increasingly complex nature of sewage and the concentration of treatment in larger con-

* Presented at the meeting of the Royal Sanitary Institute, Feb. 23, 1934, and reviewed in *The Surveyor*, 85, 285–86 (March 2, 1934).

trol works, regardless of the type adopted, demand competent operating personnel.

Relative Volumes of Sludge Production.—Although a greater volume of wet sludge is produced by the bio-aeration process, the amount of sludge (solids) is the same, except that in filters these solids are temporarily retained and later unloaded seasonally in the effluents. The objection of greater sludge disposal areas required for the bio-aeration process is reduced by the method of returning the waste sludge to the primary tanks for ultimate removal and by the employment of sludge digestion processes now rapidly gaining favor.

Comparison of Purification Efficiency.—Although the bio-aeration system may be more sensitive to rapid changes in the character and strength of the crude sewage, such sensitiveness has its advantages in that (1) recovery from the ill effects of biological disturbances is more rapid than in filters and (2) increase in the efficiency of control is induced. The ill effects of intermittent discharges of trade wastes can be greatly minimized by balancing such flows over longer periods of time.

Operating Results of the Chesterfield Bio-aeration Plant.—Detailed results are presented to prove that purification has been economical, effective and free from nuisance at Chesterfield, regardless of adverse conditions such as general temperature, prolonged periods of drouth resulting in strong septic raw sewage and the high percentage (33 per cent) of trade wastes such as spent gas liquors, textile, spent acid, tannery, brewery and dye wastes. The average annual percentage of purification effected (presumably based on the reduction of oxygen required as determined by the four hour oxygen absorbed test) was for three successive years 90.0, 91.9 and 92.8 per cent. Cost figures are also presented and discussed.

J. K. HOSKINS

EXPERIMENTS AND MEASUREMENTS IN CONNECTION WITH THE PURIFICATION OF SEWAGE *

BY F. C. VOKES

Various experimental studies made at the Birmingham Sewage Works in connection with operating problems are outlined in this article. The paper is divided into several sections, each of which deals with an entirely different subject.

Flow Measurement.—The volume of sewage including, if practicable, storm water, flowing to a new plant was to be measured. The present

* Presented at meeting of the Birmingham Association of Students of the Institution of Civil Engineers and published in *The Surveyor*, **85**, 209–211 (Feb. 9, 1934).

maximum is 12 m.g.d. which in future may be 60 m.g.d. and the minimum 3/4 m.g.d., all to be handled by one meter now and eventually not more than two meters. Other conditions included minimum loss of head and metering of the flow before removal of detritus. Means of measurement considered were, weirs, Venturi flume with level invert and free discharge, Venturi flume with differential measurement and Venturi tube with rectangular throat and level invert. Consideration of the limitation of each of these methods led to the selection of two Venturi tubes, one large and one small, the large to be blanked off until a later date.

Sedimentation Tanks.—Studies of the efficiency of upward flow sedimentation tanks differing in details of design indicated certain features, which when provided, increased the effective value of solids removal by about 25 per cent. Sewage admitted to the tanks through a bend turned vertically downwards with a horizontal baffle under the inlet bend and a vertical down draft tube extending from the water surface down to the sump was the arrangement giving the best result. Also tanks provided with four weirs produced a better effluent than those provided with only one weir. Deterioration of the effluent resulted when sludge was allowed to accumulate in the tanks.

Power from Sludge Gas.—The power obtainable from digesting sludge was measured directly by charging a tank periodically with sludge and observing the time in hours a gas engine of known indicated horse power would run on the gas evolved, the engine being restarted whenever sufficient gas had accumulated. Observations of the running time of the engine at various periods of the digestive process gave the following:

Time in days after sludge added	Period of gas generation, hours	Engine running hours
3	47	17¼
11	41	8
19	42	2¾
27	42	1¼

During a twelve month period 94 cubic yards of sludge had provided energy equivalent to 1.61 l.h.p. continuously. Based on these experiments it was possible to estimate the power obtainable from the entire sludge output of the sewage treatment works. Gas engines totalling 950 b.h.p. have since been installed and further extensions are proposed.

Heating Digested Sludge.—The effect of sludge heating on gas generation in a digestion tank was studied using for the purpose an uninsulated tank half above ground level 33 by 16.5 ft. in plan and 14 ft. deep. Heat was applied mostly by means of steam injected into the sludge. The average sludge temperature during nine months was 74°

F. while the corresponding atmospheric temperature was 50° F. The gas evolved was more than twice the average quantity obtainable from the smaller volume of sludge dealt with in the same tank at an average temperature of 52° F. The total heat units applied to the sludge during the nine months were 53 per cent of the total heat units in the gas evolved. Hence the effective value of the plant has been increased by the heating by over 100 per cent.

Further studies were undertaken to determine the heat losses to be expected from large open sludge digstion tanks and the possibility of maintaining summer temperature in the digesting sludge during the winter months by employing the waste heat from the gas engines of the Birmingham plant. By observing temperatures during nine weeks in winter in two similar tanks the sludge in one of which was heated by steam raised in a portable boiler, it was found that the 2¼ million gallons of heated sludge was 9° F. warmer than the unheated sludge. Estimating the gas generated and the proportion of waste heat recoverable from gas engines consuming this amount of gas it was proved that the sludge could be maintained at a considerably higher temperature than that of the winter atmosphere.

Pumping Sludge Through Long Mains.—Sludge containing 92 per cent of water is pumped through a 12-inch main 3.34 miles in length. Water would flow through this main by gravity but the sludge requires forcing through by a three-throw ram pump under a pressure of 50 lbs. per square inch. The loss of head due to friction when transporting this sludge at a velocity of 1.1 ft. per second is 12 or 13 times, therefore, that which would occur if the pipes were new and clean and conveying water.

In another case, sludge containing 92 per cent water is pumped through a 12-inch new main, ¼ mile long. The velocity is 1.9 ft. per second, and the pressure at the pump is 10 lbs. per square inch. The loss of head in this instance is about 2½ times that which would occur in conveying water. In still another instance, sludge containing 96 per cent water pumped through a 12-inch main, 3700 feet long at 1.75 ft. per second followed by 150 feet of 9-inch main at a velocity of 3.1 ft. per second results in a loss of head of 31 feet compared with a loss of 6 ft. if the liquid were water instead of sludge.

Tests of Air Compressors.—Procedures are given that were employed in testing various makes of air compressors considered for use in the activated sludge plants of the city.

J. K. Hoskins

EFFECTS OF GAS LIQUOR ON SEWAGE PURIFICATION *

By Walter Scott

From 1926 until 1932 the spent gas liquors from the local gas works have been discharged to the sewers of Bury and this mixture of sewage and spent liquors has been treated in the sewage purification plant. Since 1932 the crude ammoniacal liquors have been so disposed of owing to the poor market for recovered by-products. A steadily increasing deterioration in the quality of the sewage works effluent has resulted, the oxygen absorbed figure having doubled in six years. These ammoniacal liquors impart a characteristic brown color to the sewage throughout the treatment process and have caused the percolating filters to lose much of their bacterial growth.

The author considers that as little as 0.35 per cent of gas liquor in the sewage is serious and that 2 per cent of either spent or untreated liquor causes heavy increase in expenditure for satisfactory purification. The color results from the oxidation of the higher tar acids such as catechol and quinone which, after oxidation, are still organic and absorb large amounts of oxygen from permanganate.

Experiments are described in which activated carbon was employed to adsorb the phenols from the spent gas liquor. Filtration through powdered animal charcoal removed at least 86 per cent of the oxygen absorbing quality of such liquors. Later, in Germany it has been found that the phenols are adsorbed from gas liquors by filtration through carbon filters 16 feet thick. As the adsorption efficiency decreases, the carbon is regenerated by dissolving out the phenols with hot benzol and removing the benzol-phenol liquor by steaming. The benzol is recovered by distillation, the oily residue from which contains the phenols. At Bruchstrasse, Germany, during a run of several months, carbonizing from 1200 to 1500 tons of coal per day, 93.5 to 96.0 per cent of the phenol was removed from 11 million gallons of ammoniacal liquor, using 4 tons of carbon. Carbon for replacement amounted to 1.3 tons.

The experiences with the spent gas liquors at the Bury sewage works are related in some detail, where the treatment consists of detritus removal, coarse screening, 12 hours of sedimentation followed by either Simplex surface aeration or oxidation on percolating filters. Periodic deterioration and gradual recovery of efficiency was a common experience both when spent liquor was either added to or kept out of the sewage. Following the inclusion of spent gas liquors in the raw sewage, the volume treated by the surface aeration plant had to be reduced from

* Presented at a meeting of the North Western Branch of the Institute of Sewage Purification, March 9, 1934, and reviewed in *The Surveyor,* **85**, 327, 333–34 (March 16, 1934).

1 m.g.d. to 0.6 m.g.d., and the power increased from 20 to 34 h.p. per million gallons, yet the effluent was distinctly inferior to that obtained before the spent liquor was admitted. Following the closing down of the sulphate recovery plant the crude ammoniacal liquors are added to the raw sewage with even more serious detrimental effects on the sewage purification process and on the filter beds.

Discussion.—Discussion of the paper brought out similar troubles at other sewage treatment plants resulting from the discharge of gas works wastes to sewers. Dr. W. Watson had tried some experiments at Burnley with activated carbon for removal of phenols, but the particular carbon he used had been used up after continuous runs of about one day.

Mr. F. W. Allen of Bolton confirmed the observation regarding the gradual recovery of biological efficiency following the continuous application of toxic gas liquors to the influent sewage of treatment plants. He considered the sulphocyanide content as a more reliable index of the strength of gas works wastes than the oxygen absorbed test and had noted the gradual decrease in the content of sulphocyanide in sewage effluents as the bacteria became accustomed to these toxic products contributed by gas works liquors. He considered the cost of activated carbon treatment to be very high because of the high price of the carbon.

J. K. HOSKINS

PROPOSED SEWAGE DISPOSAL PLANT FOR THE CITY OF TORONTO *

Canadian Engineer, **66**, 5 and 10 (January 30, and March 6, 1934)

A $25,000,000 project for intercepting sewers and sewage treatment works for Toronto was noted briefly in *The Canadian Engineer*, November 28, 1933, with an abstract in THIS JOURNAL (**6**, 1, pp. 142). The present abstract is based upon details as carried in the issues cited and in the printed report of the consulting engineers.

The project includes new intercepting sewers; a high service pumping station; a force main and outfall sewer to the plant; sewage treatment works of the activated sludge type and an outlet conduit to Lake Ontario. The recommended treatment site is known as the Highland Creek site. It is located near the lake front about 9 miles east of the center of the city. The new plant will ultimately treat all of Greater Toronto sewage except that delivered to the existing North Toronto works. The present Eastern Avenue tanks are to be abandoned.

Sewage flows are estimated at

* Staff article from report by Gore, Nasmith & Storrie and Metcalf & Eddy, Consulting Engineers.

	1939	1948	1970
Minimum	72 cfs.	85 cfs.	130 cfs.
Average	144 "	170 "	260 "
Maximum	360 "	425 "	647 "

Intercepting Sewers

The intercepting sewers are high level relief sewers for Toronto. Intercepting sewers for a Greater Toronto Sewerage District are deferred construction.

Pumping Station

The new pumping station will supplement an existing low level station (at Eastern Avenue). Both new and old stations will have mechanically cleaned racks. Screenings will be ejected to the high service force mains. The pumps will be motor driven.

Outfall Sewer

The outfall will consist of 3,500 lin. ft. of twin 7 ft. reinforced concrete pipe and of about 56,200 lin. ft. of 12 ft. concrete conduit. The twin pipes will be pressure pipes. The 12 ft. conduit will be a gravity conduit for flows up to 434 cfs. Beyond this capacity the upper end will be surcharged (44 ft. at 1970 max. flow). For gravity discharge Kutter N was taken at 0.014; for pressure conditions $N = 0.013$.

Much of the 12 ft. sewer will be in tunnel. Shafts at 7,000 ft. intervals are planned as permanent shafts.

Sewage Treatment Works

The plant is designed for 1948 conditions with an average flow of 105 m.g.d. Preliminary treatment is to be given to 2½ times the average flow and complete treatment to 1½ times the average.

Racks will have a capacity of 262.5 m.g.d. Bars will be spaced 1 inch. The racks will be cleaned mechanically. Screenings will be burned in a gas-fired incinerator.

Grit Chambers are designed for 1 minute detention and 1 f.p.s. velocity.

Preliminary Sedimentation Tanks will have 2 hours capacity at average rates. Sludge will be removed mechanically and oil and grease skimmed.

Aeration Tanks are planned for 5½ hours detention at 120 per cent of the average rate of sewage flow. The tanks will be spiral flow with diffused air.

Sludge Settling Tanks will provide 2.9 hours detention for the mixed

liquor and a settling rate of 650 gals. per sq. ft. per day. Sludge will be removed mechanically.

Sludge Digestion Tanks will provide 50-day digestion capacity. The sludge is estimated at 0.725 m.g.d. Ave. (1.47 max.) from the pre-settling tanks and 0.81 m.g.d. from the sludge settling tanks, both as of 1948. The tanks will be equipped with hot water coils and heated to 80–85 deg. F. Gas will be collected and used for heating the digestion tanks, for incinerating screenings and for miscellaneous heating. A possible deficiency of gas during winter months is suggested.

Sludge Dewatering will be by vacuum filters. 400 cu. yds. of 80 per cent moisture filter cake per day are anticipated for 1948 (as filtered from 0.244 m.g.d. of digested sludge). Dewatered sludge will be disposed of as fertilizer or fill.

Plant elevations are shown thus:

Flow Line at Racks	Elev. 271.0
Flow Line at Grit Chambers	270.0
Flow Line at Preliminary Sedimentation Tanks	267.
Flow Line at Aeration Tanks	265.
Flow Line at Final Settling Tanks	263.5
Maximum Lake elevation is	249.

Outlet Conduit

Treated sewage will be delivered to a control chamber which will connect to a 9¼ ft. down-take shaft 115 ft. deep. The outlet conduit will have a capacity of 262.5 m.g.d. It will be 9¼ ft. in diameter, 4,480 ft. long, will be located 91 ft. below lake bottom and will terminate in a 9¼ ft. uptake shaft in 34 feet of water. From the uptake shaft will lead tapered diffusion pipes about 300 feet long, with outlet ports in the top of the pipes. Partially treated storm flow will be discharged through the submerged outlet. If further outlet capacity is needed, the effluent from the activated sludge plant will be discharged over a spillway at the control chamber and so to Highland Creek.

Estimates of Cost

The costs of construction are estimated thus:

New Intercepting Sewers		$ 5,411,000
Pumping Station		
Substructure	$ 184,000	
Superstructure	128,000	
Pumps and Motors (4)	132,000	
Electric Control and Power Wiring	44,000	
Valves and Piping	118,000	
Miscellaneous Equipment	92,000	
Venturi Meters and Conduits	83,000	
Administrative, engineering, legal	78,000	
	$ 859,000	$ 859,000

Outfall to Plant		
3,500 l.f. 7 ft. Twin Pipe	$ 495,000	
12 ft. Sewer	8,021,000	
Land, R/W, engineering, etc.	858,000	
	$9,374,000	$ 9,374,000
Sewage Treatment Works		
Racks and Venturi Meters	$ 160,000	
Grit Chambers	80,000	
Preliminary Sedimentation Tanks	800,000	
Aeration Tanks	1,440,000	
Final Sedimentation Tanks	1,465,000	
Administration and Laboratory Building	115,000	
Air compressing and return sludge pumping	555,000	
Main Conduits	250,000	
Grading and Landscaping	190,000	
Exterior Lighting	145,000	
Water Supply	60,000	
Miscellaneous Equipment	50,000	
Preparation of Site	810,000	
Digestion Tanks	1,650,000	
Sludge Dewatering Plant	330,000	
Sludge handling and disposal equipment and facilities	150,000	
Land, Administration, Engineering, etc.	1,185,000	
	$9,435,000	$ 9,435,000
Outlet Conduit		
Conduits to Control Chamber	$ 9,000	
Overflow Channel	21,000	
Control Chamber	51,000	
Entrance Shaft	11,000	
Tunnel	385,000	
Discharge Shaft	111,000	
Diffusion Pipes	103,000	
Administration, etc.	69,000	
	$ 760,000	760,000
		$25,839,000

A six-year construction program is planned.

FRANK C. TOLLES

LOSS OF HEAD IN ACTIVATED SLUDGE AERATION CHANNELS

By Darwin Wadsworth Townsend

Proceedings American Society of Civil Engineers, 60, 63 (Jan., 1934)

This paper presents the results of studies made to determine Kutter "*n*" values appropriate to channels which carry a mixture of sewage and activated sludge and into which air is introduced through diffuser plates.

The author finds (1) that air diffusion increases the resistance to flow and (2) that resistance increases as velocity decreases. Resulting changes in resistance to flow have been construed into varying values of "*n*" and referred also to "*n*" of 0.013 as a base representative of usual flow conditions.

An experiment was conducted in the feed channels at Milwaukee. The sewage contained 3,000 p.p.m. of suspended solids and was agitated with diffused air at the rate of 1 cu. ft. of free air per diffuser plate per minute. The channel was 8 ft. wide with a wetted cross-section of 75.39 sq. ft.; a wetted perimeter of 27.47 ft. and a mean hydraulic radius of 2.774 ft. The channel was equipped with two rows "practically continuous" of diffuser plates. Head recorders were 123.34 ft. apart.

Graphical records of head at the two stations showed continuous pulsations of between 0.02 and 0.05 ft. at the upstream station and between 0.06 and 0.09 ft. at the downstream station. The rate at which the air bubbles coalesce is regarded as having an important effect upon the pulsations. The observed velocities of lineal flow were between 0.9 and 1.3 f.p.s.

Averaged head curves at the two stations showed a consistent difference of 0.02 to 0.03 ft. A table gives the relative values of the hydraulic properties of the aerated channel when referred to an unaerated channel. In general it is found that if the resistance due to aeration were removed and the channel restored to an "*n*" of 0.013, the velocities would be increased in percentages between 270 and 302. Equivalent percentages of increase over normal (*i.e.*, those appropriate to $n = 0.013$) are given for the wetted perimeters and for the mean radii.

In order to bring the data within the range of velocities which occur in aeration tanks, the plotted curve ("*n*" against velocity) which results from velocities between 1.3 and 0.9 f.p.s., was extended and found to be asymptotic to zero velocity with an "*n*" value of 0.23. The curve is for a ridge and furrow system of aeration. A similar curve for circulatory aeration is suggested and shown.

Readings from the curves as plotted (Fig. 5 of the original paper) are as follows:

Relationship between Value of n and Flow Velocities

Velocity f.p.s.		Values of Kutter n	
		Ridge and Furrow	Circulatory
1.3	Observed	0.034	0.030
1.2	Observed	0.034	0.031
1.0	Observed	0.039	0.035
0.9	Observed	0.032	0.036
0.8		0.047	0.043
0.6		0.061	0.056
0.4		0.080	0.077
0.2		0.112	0.108
0.1		0.140	0.134

Discussion. By H. L. Thackwell, *Proc. Am. Soc. C.E.*, **60**, 429 (March, 1934).

It is pointed out that the resistance to flow of the curve data was computed from the Chezy formula to involve the second power of the mean velocity, but that the curves have been extended to limits below the critical velocity and wherein the same relationship may not obtain. Also it is suggested that had allowance for the jet action of the admitted air been made through a reduction in the net cross-section of flow it would have been found that "n" increased with decrease of velocity, although nearly the same for all values of V.

Collaterally, the commentator reports original experiments to obtain data relative to the behavior of air bubbles in water. In partial conclusion he states that aeration may be steady, pulsating or turbulent; that for the first two conditions the percentage of air is less than seven by volume, the bubbles are all larger than 1 mm. and the upward velocity is non-accelerating at 0.8 f.p.s.; that, in turbulent flow, the upward velocity of the bubbles is non-accelerating and proportional to the percentage of air—ranging from 0.8 to 2.0 f.p.s.; and that the percentage of injected air is proportional to the submergence with the most efficient aeration at 50 per cent submergence.

FRANK C. TOLLES

SOUNDNESS TESTS FOR SEWAGE FILTER MEDIA

BY H. G. PAYROW

Civil Engineering, **4**, 90 (Feb., 1934)

According to a tentative procedure established by a committee of the Sanitary Engineering Division, American Society of Civil Engineers, the soundness of materials for sewage trickling filter media is

determined by alternate immersions of specimens in sodium sulphate solution followed by a period of drying. The crystallization of the absorbed sulphate tends to disrupt the material and thus simulates the action of freezing. Repeated cycles of hydration and dehydration produce the effect of alternate freezing and thawing to which the material is subject in trickling filters.

Variations in the technique of applying the sodium sulphate test may cause wide variations in results. The solubility curve for sodium sulphate changes rapidly between temperatures of zero and 30° C. and thus affects the strength of the solution unless constant temperature is obtained (the author suggests 25° C. as standard). The article is devoted to a comparison of the sodium sulphate test and to an alternative freezing and thawing test. Comparison was made by subjecting similar specimens to 20 cycles of the sodium sulphate test, and 100 cycles of freezing and thawing, a cycle in each case consisting of 24 hours.

In result of the tests the author concludes that 20 cycles of sodium sulphate are probably more than sufficient as a soundness test, and that 100 cycles of freezing and thawing are equally effective. The sodium sulphate test is apparently the more severe of the two, and it produces debris which has a consistently smaller uniformity coefficient than the debris from the freezing and thawing test. The author also concludes the freezing and thawing test is not only conclusive and closest to actual conditions but is the most convenient and probably the most economical for routine testing purposes. He believes that a complete time cycle of 8 hours for freezing and thawing would be sufficient. He further suggests the use of an adapted commercial electric ice freezer with shallow trays which are equipped with a heating element and with the cycle controlled by means of an electric time switch.

(*Abstractor's Note:* The procedures of the Sanitary Section Committee for the sodium sulphate test are contained in *Proc. Am. Soc. Civil Engineers*, Sept., 1929, p. 1839. They provide for making up the sodium sulphate solution at a temperature between 29° and 35° C. and for maintaining it at or near 21° C. during the entire test.)

FRANK C. TOLLES

RAINFALL STUDIES FOR NEW YORK, N. Y.

BY S. D. BLEICH

Proceedings American Society Civil Engineers, **60**, 157 (Feb., 1934)

This paper derives rainfall intensity curves for use in sewer design. The particular data are in terms of the 62-year rainfall record of New York City. The methods of analysis are applicable generally.

Involving the three variables of intensity, duration and frequency three types of formulae are presented wherein the frequency factor is

constant. These several forms are:

(*a*) Reciprocal Formula: $R = \frac{A}{t + b}$

(*b*) Exponential Formula: $R = \frac{B}{t^c}$

(*c*) Modified Exponential Formula: $R = \frac{C}{(t + d)^e}$

R = Rainfall Intensity—inches per hour
t = Duration of Storm—minutes
A, B, C, c, and d = Constants for each frequency of storm

The paper gives the formulae evaluated in terms of New York data and compares the results from the standpoint of the average difference between the rainfall rate obtained from the formula and the rainfall rate actually observed. This comparison, inclusive of an old formula which was derived by trial and error, is shown by the following quotation of Table 8 of the paper.

TABLE 8.—*Comparison of Various Formulae for Rainfall with Observed Data*

	Two-Year Storm		Ten-Year Storm	
	Formulae $R =$	Ave. Diff. from Observed Data Inches/ Hour	Formulae $R =$	Ave. Diff. from Observed Data Inches/ Hour
Old Formulas	$\frac{120}{t + 20}$	0.285	$\frac{150}{t + 15}$	0.387
(*a*) Reciprocal Formulas	$\frac{96.3}{t + 15}$	0.140	$\frac{168}{t + 20}$	0.182
(*b*) Exponential Formulas	$\frac{12.38}{t^{0.53}}$	0.144	$\frac{18.1}{t^{0.53}}$	0.370
(*c*) Modified Exponential Formulas	$\frac{41.62}{(t + 9)^{0.813}}$	0.096	$\frac{63.75}{(t + 12)^{0.795}}$	0.155

The author also develops an expression $Rn^h = K$ involving intensity (R) and frequency (n) and constants h and K. Likewise he investigates the probability method.

Conclusion is reached that the best and most expedient formulae for sewer design are those labelled "Modified Exponential."

The formulae given are derived from observed data by the method of least squares. An appendix to the paper shows the calculations.

FRANK C. TOLLES

GAS HAZARDS IN SEWERS AND SEWAGE TREATMENT PLANTS

BY R. R. SAYERS

Public Health Reports (U. S. P. H. S.), **45.5**, 145–155 (Feb. 2, 1934)

Gas hazards in sewers and sewage treatment plants are due to inflammable and poisonous gases and to oxygen deficiency. Inflammable and poisonous gases may be derived from: (1) low volatile liquids entering as part of sewage; (2) leakage from gas mains into sewers, or (3) products of fermentation or digestion of sewage.

Inflammable or poisonous gases found at treatment plants are methane, hydrogen, carbon dioxide, carbon monoxide and hydrogen sulphide. In a table it is shown that in gases from sludge digestion tanks oxygen varies from 0 to 1.2 per cent, CO_2 from 3 to 30 per cent, H_2S from 0 to 0.1 per cent, hydrogen from 0 to 8.2 per cent and methane from 63 to 84 per cent.

Concerning the properties of gases found in sewers and sewage treatment plants, the following information supplemented by detailed text, is given. (Numbered columns give data indicated below):

	(1)	(2)	(3)	(4)	(5)	(6)
Ammonia (NH_3)	16–27	−35	—	.25–.45	.03	.01
Benzol (C_6H_6)	1.4–8.	80.2	—	—	.31–.47	.15–.31
Carbon dioxide (CO_2)	—	−78.2	30	6–8	4–6	2–3
Ethane (C_2H_6)	3.2–12.5	−93	—	—	—	—
Methane (CH_4)	5.0–15.	−164	—	—	—	—
Gasoline (C_6H_{14} to C_7H_{16})	1.4–6.	50–140	—	2 –2.5	.1–.3	0.1
Hydrogen Sulphide (H_2S)	4.3–46.	−60.2	.06–.10	.05–.07	.02–.03	.005–.01
Sulphur dioxide (SO_2)	—	−10	.2	—	—	.01
Carbon monoxide (CO)	12.7–74.	−192	.5–1.0	.2–.3	.05–.10	.02

Key to numbered columns:

(1) Inflammable limits in percent by volume.
(2) Boiling point, ° C.
(3) Percent fatal in 30 minutes or less.
(4) Percent causing dangerous illness in ½ to 1 hour.
(5) Percent that can be borne without serious effects for ½ to 1 hour.
(6) Maximum safe concentration, percent.

Preventive Measures.—A manhole should never be entered until assured that it is free from dangerous gases, by testing with an approved

lamp, by ventilation, or testing for the presence of poisonous or explosive gases. Odors are important indicators, though some gases (CO and H_2) have little or no odor. Inflammable gases may be detected by the Burrell methane indicator, the Martienssen methane detector and the V. C. C. methane detector. Oxygen deficiency and poisonous gases may be shown by using small animals, especially birds (canaries are best). Portable apparatus has been developed for testing explosiveness.

Sewage tanks should be well ventilated before entering them. Several types of portable blowers with explosion-proof motors may be used for ventilation, which should be continued during the time workmen are in a tank. Where digestion gases are collected and used for heating, air may leak into the distribution system if the plant is shut down or gas generation stopped. To prevent infiltration of air, the entire system can be kept under a few inches of water pressure. If air is kept out of the system, explosions will not occur. For pipes likely to have air leakage, release diaphragms augmented by water seals and screens are better than screens alone, which are often inadequate.

If it is necessary to enter an atmosphere containing noxious gases with insufficient ventilation, the hose mask is recommended, especially for man holes. The type N canister mask is also satisfactory when the air contains only a small proportion of poisonous gases and sufficient oxygen to support life. The hose mask has the advantage that nothing has to be changed until the parts are worn out. It may be used at 100 ft. distance, but longer hose is not usually practicable. The hose mask with hand-operated blower delivers air at the face piece in excess of that necessary to be breathed. If the hose should leak, the leak will be outward rather than into the hose. It is obvious that the intake end of the hose mask should be in fresh air.

Treatment of Poisoning

1. The victim should be removed to fresh air as soon as possible.

2. If breathing has stopped or is weak and intermittent, or present in but occasional gasps, artificial respiration by the Schaefer method should be given persistently until after breathing is resumed, or until after the heart has stopped.

3. Circulation should be aided by rubbing the limbs and keeping the body warm with blankets, hot water bottles, hot bricks or other devices, care being taken that these are wrapped or do not come in contact with the body and produce burns. Other stimulants such as hypodermics of caffein, sodium benzoate or camphor oil, should not be administered except by a doctor after he has considered the possibility of over stimulation and consequent collapse.

4. The patient should be kept at rest, lying down in order to avoid

any strain on the heart. Later he should be treated as a convalescent and given plenty of time to rest and recuperate.

5. After effects of poisoning by gases should be treated symptomatically.

6. It should be emphasized that inhalation for a period of 20 to 30 minutes, of oxygen or a 5 per cent mixture of carbon dioxide in oxygen if available will, when given immediately, greatly lessen the number and severity of symptoms of carbon monoxide poisoning, as well as decrease the possibility of serious sequelae. All industries in which this type of poisoning commonly exists, should provide inhalers for the efficient administration of these treatments. This apparatus should be placed at convenient points and employees should be trained in its use so that resuscitation may be effected immediately.

H. W. STREETER

CHEMICAL PURIFICATION AT FREEPORT, LONG ISLAND

BY L. L. LUTHER

Public Works, **65**, 20–22 (Jan., 1934)

The sewage treatment works at Freeport, a residential town of 20,000, consists of sedimentation works, separate sludge digestion by stages, covered sludge beds and effluent chlorination.* During the past year experiments have been initiated in chemical treatment, using ferric chloride as a precipitant. The results in part are stated as follows:

1. A great improvement in the quality of effluent,
2. An acceleration in the speed of sedimentation,
3. The degree of purification may be varied widely with the amount of chemical added,
4. The efficiency of the treatment was greater with proper mixing, which is important,
5. Prechlorination required no more chlorine than postchlorination,
6. Improvement in the condition of the tanks was noted with ferric chloride and prechlorination treatment (less floating solids and cleaner tanks),
7. The optimum time of mixing was about 15 minutes,
8. Ferric chloride accelerated dewatering of fresh and digested sludge,
9. The cost of chemical treatment varies with the degree of treatment, ranging from 80 cents per m.g. with an effluent B.O.D. of 120 p.p.m. to $10.00 per m.g. with an effluent B.O.D. of 60 p.p.m.

For an ultimate sewage flow of 8.63 m.g.d. the estimated cost of buildings and appurtenances for chemical treatment is $11,200 and for a

* See THIS JOURNAL, **4**, 221 (1932).

mixing basin $17,500; a total of $28,700. The estimate installation cost per m.g.d. capacity ranges from $8,850 for 2 m.g.d. capacity to $3,588 for 8 m.g.d. The operating cost for the chemical treatment ranges from $2.50 to $11.70 per m.g.d., for a 2 m.g.d. plant. The use of chemical treatment at Freeport will permit utilizing a large part of the present works for many years to come, with almost any degree of treatment, it is stated, for a corresponding expenditure of money.

The experimental results obtained are illustrated by four interesting charts.

H. W. STREETER

SEWERAGE COMPANY HELD NOT LIABLE FOR DAMAGES TO LAND RESULTING FROM PERMANENT NUISANCE WHERE NUISANCE HAD EXISTED BEFORE PURCHASE OF LAND BY PLAINTIFF

Public Health Reports, **48**, No. 46 (Nov. 17, 1933)

On appeal against an adverse decision to the Texas Civil Court of Appeals, the decision of the lower court against the Bowie Sewerage Company was reversed. The court held that the plaintiff purchased land already depreciated in value and the right of action, if any, was vested in the owner of the land at the time when the pollution of the stream by reason of the flow of sewage from the septic pool across the land was first apparent.

R. W. KEHR

CITY ORDINANCE REQUIRING WATER CLOSETS WHERE SEWER CONNECTIONS WERE POSSIBLE UPHELD

Public Health Reports, **48**, No. 35 (Sept. 1, 1933)

The Missouri Supreme Court upheld decisions by the lower courts that the City of St. Louis was entitled to recover penalties for alleged violations of section 2928 of its 1926 Revised Code, which provided that whenever connection with a sewer was possible, all privy vaults should be filled and a water closet provided for each family, either in the house, or on the lot on which the house was located.

R. W. KEHR

LIABILITY OF CITY FOR SEWAGE POLLUTION OF STREAM

Public Health Reports, **48**, No. 24 (June 16, 1933)

In a suit against the City of Harrisonville, Mo., the W. S. Dickey Co. obtained $4000 damages and an injunction to prevent the city from dis-

charging the effluent from an Imhoff tank where it flowed through pasture land owned by the plaintiff company. On appeal by the City, the district court eliminated $3500 of the damages, and on further appeal by the City, the U. S. Supreme Court reversed the decision since it worked a disproportionate hardship on the City and substantial redress could be obtained by payment of money. The case was remanded to the district court for further proceedings to determine the depreciation in value of the land and to enter a decree withholding an injunction if such sum be paid within the time to be fixed by that court.

R. W. KEHR

COMPLIANCE BY MUNICIPALITY WITH STATE HEALTH DEPARTMENT'S ORDER DESIGNED TO CORRECT SEWAGE POLLUTION OF STREAM

Public Health Reports, **48**, No. 17 (April 28, 1933)

In defending an action by the State health director to obtain a writ of mandamus to compel compliance with a previous order for stream pollution abatement, the City of Van Wert, Ohio, claimed that it had no funds with which to carry out the order and could not levy taxes to do so without violating certain statutory and constitutional limitations. The Supreme Court held, however, that "Current expenses must be secondary to levies to meet mandatory requirements, such as discharge of bonded indebtedness, interest thereon, and also compliance with orders of the State department of health issued under the general State laws, which we hold to be mandatory. . . ."

The court further held that if sufficient funds were not available within the 15 mills limitation, funds must be obtained for current expenses as provided by the statutes.

R. W. KEHR

AKRON SEWAGE WORKS

BY A. B. BACKHERMS

Municipal Sanitation, **4**, 161–163 (May, 1933); **4**, 189–191 (June, 1933) and **4**, 229–232

The Akron Sewage Works, a detritus-Imhoff installation, was put into operation in December, 1928. During the first year short-circuiting and formation of grit mounds on the bottom of the tanks caused trouble which was remedied by redesigning the rakes and by proper baffling. Grit is removed from the channels by endless chain scrapers. Clogging of the slots in the revolving drum fine screens was overcome by removing

the screens from service for a week at a time and scrubbing them with a broom and water.

The following table shows the average moisture and volatile content of the grit and fine screenings for the first three years of operation:

	Grit		Fine Screenings	
	Moisture Per Cent	Volatile Per Cent	Moisture Per Cent	Volatile Per Cent
1929	70.0	64.0	88.0	65.0
1930	71.3	54.0	88.9	71.7
1931	66.5	39.4	91.1	71.7

Grease is swept upon draining beds and drained material is finally disposed of on a sludge dump along with the grit and screenings.

Removals by the various detritus units are given below:

	Flow		Rocks		Fine Screens		Detritus Tanks		Grease Beds	
	Total	M.G.D.	Cu. Yds.	Cu. Ft. per M.G.	Cu. Yds.	Cu. Ft. per M.G.	Cu. Yds.	Cu. Ft. per M.G.	Cu. Yds.	Cu. Ft. per M.G.
1929	7113	19.7	106 6	0.40	301.1	1.14	906.7	3.4	63.1	0.24
1930	9888	29.6	156.3	0.44	540.2	1.61	1841.1	5.4	57.9	0.14
1931	10210	28.1	159.0	0.42	624.9	1.64	2145.4	5.65	60.1	0.16

The return of humus sludge to the Imhoff tanks caused scum formation, and when this difficulty could not be overcome the return of the sludge was abandoned. In 1929 the average removal of suspended solids by the Imhoff tanks was 57.8 per cent, B.O.D. 16.2 per cent; 1930 suspended solids 55.4 per cent, B.O.D. 28.1 per cent; 1931 suspended solids 53.8 per cent, B.O.D. 27.4 per cent. The amounts of sludge removed (dry basis), moisture, volatile and pH for 1929 are 795 tons, 85.3 per cent, 48.9 per cent and 5.9 respectively; for 1930 they were 8870, 89.1, 50.9 and 6.4; for 1931 they were 9531, 93.0, 52.5 and 6.7. The total operating cost for the Inhoff units was 0.52 per m.g. in 1929; 0.67 in 1920 and 0.64 in 1931.

The effluent from the Imhoff tanks flows through dosing chambers onto trickling filters. Trouble caused by pooling was remedied by the application of chlorine as a gas or as a hypochlorite in normal or con-

centrated form. The average removal of B.O.D. by the filters for the three years was 81 per cent. The effluent contained from 2.1 to 3.2 p.p.m. of nitrate nitrogen and had an average dissolved oxygen content of 5.1 p.p.m. The cost of operating the filters was 0.40 per m.g. in 1929; 0.53 in 1930; and 0.42 in 1931.

Sludge is applied to drying beds with a moisture content of 92.0 per cent and removed with a moisture content of 47.0 per cent.

The overall efficiency of the plant for the three years of operation is indicated by the per cent removal of suspended solids and B.O.D. which is as follows:

	Suspended Solids	B.O.D.
1929	84.8	89.0
1930	75.0	70.5
1931	73.5	71.5

The operating costs for the plant are shown below:

	1929 per m.g.	1930 per m.g.	1931 per m.g.
Treatment units	4.79	4.48	4.93
Ground upkeep	3.32	2.37	1.49
Total	8.11	6.85	6.42

E. HURWITZ

FROM TRICKLING FILTER TO DUNBAR TYPE

BY EARNEST W. STEEL

Municipal Sanitation, **4**, 301–302 (Sept., 1933)

The town of Newasota, Texas, has rebuilt its unsatisfactory trickling filter plant into one of the Dunbar type. The old trickling filter was made over into a Dunbar bed retaining its original dimensions, 50 ft. by 50 ft. and two new beds were constructed each 35 feet by 100 feet. The filtering material consists of 24 inches of 3 to 6 inch stone, 12 inches of ½ to 1½ inch gravel, 6 inches of ⅛ to ⅜ inch gravel and 6 inches of ⅛ to $\frac{1}{25}$ inch sand. The beds are underdrained with one row of 6 inch drain tile. The concrete bottoms drain towards this tile.

Imhoff effluent is applied to one bed until clogging occurs. It is then turned into the second bed while the first undergoes a resting period. After several days the sludge which has dried on the surface of the bed is swept off, the surface sand raked and the bed is again ready for use.

The beds are dosed at a rate of 1,800,000 gallons per acre per day.

Clogging occurs in about two weeks. Several days are required for drying. The effluent has a suspended solid content of 22 p.p.m., pH of 7.4, and approximately 70 per cent of the B.O.D. of the applied sewage is removed. No complaint of odors has been made and the stream into which the effluent discharges shows no undesirable effects.

By the use of local labor, which could be had at one dollar and twenty-five cents per day, the plant was reconstructed at a cost of 6800 dollars.

E. HURWITZ

LOW COST SEWAGE DISPOSAL PLANT

BY R. G. UPTON

Municipal Sanitation, **4**, 304 (Sept., 1933)

A sewage plant that is both inexpensive and efficient has been operating for the past few years at Nacogdoches, Texas. The plant consists of a hopper-bottomed settling tank, separate sludge digestion and Dunbar beds. Sewage passes through the settling tank which has a depth of 16 feet onto the Dunbar beds of which there are two. Sludge is pumped into the digestion tanks two to three times daily during the summer months and only once during winter. Measured by the reduction of B.O.D. the plant has an efficiency of between 60 to 77 per cent depending on the season of the year.

E. HURWITZ

SEWAGE TREATMENT AT SHANGHAI

BY WALTER C. BUCHLER

Municipal Sanitation, **4**, 260–261 (Aug., 1933)

The sewerage system in Shanghai is divided into three sections, Western, Eastern, and Central Northern. A by-pass arrangement is provided whereby a large portion of the sewage of the Central area can be treated on the Eastern instead of Northern, and North could deal with the whole of Central. The activated sludge process is used; the effluent being discharged into the nearest river. Sludge is dried on sand beds.

Chinese labor and locally made material are used wherever possible. Two million gallons of sewage are treated daily in the Eastern section, 900,000 in the Western, and 60,000 in the Northern.

Construction difficulties due to the high water level in Shanghai are encountered. Practically all pipes lie in mud and must be protected against corrosion.

A large portion of Shanghai sewage is still disposed of as night soil.

E. HURWITZ

PREFILTRATION TREATMENT OF SEWAGE SLUDGE

By F. W. Mohlman and G. P. Edwards

Ind. Eng. Chem., **26**, 226–30 (Feb., 1934)

Since the discovery of the value of ferric chloride as a conditioner for sludge, attempts have been made to discover some augmenting or cheapening agent to use with it. This article describes experiments in which various chemical coagulants and physical filter aids were added to sewage sludge in order to condition the sludge prior to vacuum filtration on Buchner funnels.

Using ferric chloride, the optimum amount for fresh sludge varied from 5 to 6 per cent; with digested sludge, the optimum amount ranged from 8 to 14 per cent. The amount of ferric chloride required by fresh sludge was not reduced appreciably by use of sulphuric, nitric, or hydrochloric acid, and only slightly by use of chlorine. A small amount of sodium dichromate, 1 per cent or less, greatly reduced the amount of ferric chloride required. The sodium dichromate probably not only prevented the reduction of the iron and its loss in the effluent but also served as a good coagulant itself when reduced to chromic chloride. With digested sludge, a saving in ferric chloride can be effected by adding enough lime to raise the pH to 9.0 or above.

Ferrous chloride, zinc chloride, sodium chloride, and ferrous sulfate plus lime were of little value as coagulants. Aluminum chloride, chromic chloride and titanium tetrachloride were good coagulants but no better than ferric chloride. Alum, chlorinated copperas, and ferric sulphate were inferior to ferric chloride as coagulants for fresh solids. When fresh solids were heated from 72° to 90° F., before the addition of ferric chloride, a marked improvement in filtration was noted. Further heating to 122° F. hindered filtration. Coke breeze, petroleum coke and ground slag were of little value as filter aids. Large amounts of paper pulp, equal to 50 per cent or more of the weight of dry sludge solids, possessed some value as a filter aid.

P. D. McNamee

REPORT ON THE APULIAN AQUEDUCT (ITALY) 1931–1932

By Ugo Bono, M.P.

Translated by A. Calabrese

These *Proceedings* (138 pages) deal primarily with the construction of the great Apulian aqueduct, in the "heel of the boot" of Italy.

The last five pages of the report deal with the experimental plant for sewage treatment at Foggia.

Experiment Station for the Purification of Sewage

The experiment station (which has been in operation only one year) was built to study the purification and utilization of sewage.

The plan and equipment of the station comprise:

1. Mechanical purification by sedimentation.
2. Disposal of sludge.
3. Oxidation of settled sewage by modern artificial means.
4. Utilization of effluent and sludge for agricultural purposes.

The equipment at the station is capable of treating about 500 cubic meters of sewage per day, corresponding to the sewage from a city of 5,000 inhabitants.

As the sewage from the septic Foggia outfall reaches the plant it passes through a bar screen (bars 3 centimeters apart) and from here it is pumped to the sedimentation tanks.

There are three sedimentation tanks; two are of the Imhoff type with lower chambers for the digestion of sludge and the third a plain settling tank. A distributor apportions the sewage to the three tanks. Alongside these sedimentation tanks there are two separate digestion tanks. The effluent from the Imhoff tanks flows into two tanks which distribute the liquids among the various oxidation units.

Three systems have been investigated; percolating filters, contact filters (aerated colloiders) and activated sludge. Up to the present time only the first (rotary type) of the three sprinkling filters has been constructed. This functions intermittently, with a siphon, and has a surface area of about 16 square meters and a depth of about 2 meters. Two other percolating filters have been designed, one with fixed nozzles and the other a rotary sprinkler. The filter effluent is settled in final settling tanks.

The contact filters (aerated colloiders) comprise two units, one for complete treatment and the other for only partial treatment. Each unit is constructed in a rectangular tank which is divided into two compartments, an aeration compartment and a sedimentation tank. The aeration compartments contain surfaces for accumulation of biological growths. Compressed air is diffused below the surfaces by means of a swinging perforated tube.

The selection of the type of activated-sludge unit was difficult since a wide variety of designs are in use throughout the world. The type used at Essen-Rellinghausen was chosen, combining compressed air with mechanical agitation.

Of the three previously mentioned settling tanks, only two have been constructed, one of which constitutes a single tank 6.25 by 2 by 2 meters, with one hour aeration, alongside two other similar tanks for reaeration of sludge. The second group is for partial purification.

At the bottom of all of the tanks, and on one side only, there is a row of porous plates, which diffuse the air. In the center of each tank there is a horizontal bar equipped with paddles. These revolve by means of a small motor, and serve mechanically to aerate the sewage.

Another activated sludge unit is the "Simplex" type. This is a cylindrical aeration tank with a central metal pipe, attached to the top of which is a metal wheel resembling a saucer. This saucer-like wheel continuously revolves. The wheel is revolved by a small motor and as the wheel revolves it continuously mixes the activated sludge and sewage, mechanical aeration thereby being secured. A much longer period of aeration is necessary with this unit. The mixture of activated sludge and sewage is then settled in a clarification tank. A small diaphragm pump is attached to the motor that revolves the wheel, and this pump returns to the sewage some of the activated sludge which settles in the bottom of the tank.

The sludge deposited in the bottom of the final settling tanks from the activated sludge units is returned and reaerated. A portion of the return sludge re-enters in cycles, mixing directly with the fresh sewage (as in plant No. 1) or passing through the reaeration tanks (as in plant No. 2), the surplus going to a small sump, from which it is pumped and settled in sedimentation tank No. 3, finally passing to separate digestion tanks.

The effluent from the plant is chlorinated by means of the Berlin Gesellschaft chlorinator. This has an automatic control, and can supply enough chlorine water for complete sterilization of all the liquids treated at the plant.

Even though the experiment station has been in operation only one year, numerous data have been collected daily so that definite conclusions can be established for the construction and operation of purification works for sewage and for sludge suitable for fertilizer.

Operation of the plant comprises (*a*) measurement of sewage treated, (*b*) measurement of compressed air in each unit, (*c*) measurement of electrical power, (*d*) collection of typical samples of sewage and effluents from the various units, (*e*) chemical analyses of sewage and effluents in various stages of purification, and analysis of digested sludge and gases from the digestion tanks, and (*f*) bacteriological analyses of sewage and effluents.

Chemical and bacteriological laboratories are included in the plant. Research work is also conducted.

In order to study the use of sewage and sludge for agricultural purposes, two acres of land surrounding the plant have been provided. This plot is divided into two sections, one for irrigation experiments and the other for fertilizer tests.

Up to the present time the experiments for the first year and perhaps for some years to come, are of an orienting nature. The land surrounding the plant has been uncultivated for three years and has been trampled over during the building of the station so that it was almost impossible to plough it.

The land for irrigation experiments was divided into ten plots of about 800 square meters each and was treated as follows: (1) water from the Sele, (2) clarified effluent only, (3) oxidized effluents, (4) same as (1) using twice as much water, (5) same as (2), (6) same as (3), (7) same as (2) but using twice as much water, (8) same as (1), (9 and 10) no irrigation, but set aside for emergencies or special tests.

The land set aside for fertilizer tests was divided into seven different parcels of about 400 square meters each. The parcels were treated as follows: (11) manure, 18 tons per acre, (12) sludge equivalent to the organic material in the manure used in (11), sludge equivalent to the total dry material contained in manure used in (11), (14, 15, 16, respectively) same as (11), (12), (13), (17) twice the amount of sludge used in (12) and (15). All of the parcels were fertilized with superphosphate in the amount of 4,409 pounds per acre and irrigated as needed with Sele water.

It was recommended by the director of the Royal Agricultural Institute of Portici and by the professor of agricultural chemistry that corn be grown on all of the test plots.

No conclusions can be drawn as yet as to the purification obtained, as the plant has been completely operated for only a few months. The agricultural tests are not completed as the crop is not yet grown.

Other fertilizer and irrigation experiments which were made on a much smaller scale have been of an orienting nature, using garden plants such as tomatoes, egg plants, melons and peppers. Very good results have been obtained.

The effect of sludge for fertilization on certain ornamental plants and flowers has been tried on a parcel of 300 square meters. The flowering started about the middle of June, and has continued vigorous and abundant.

H. GLADYS SWOPE